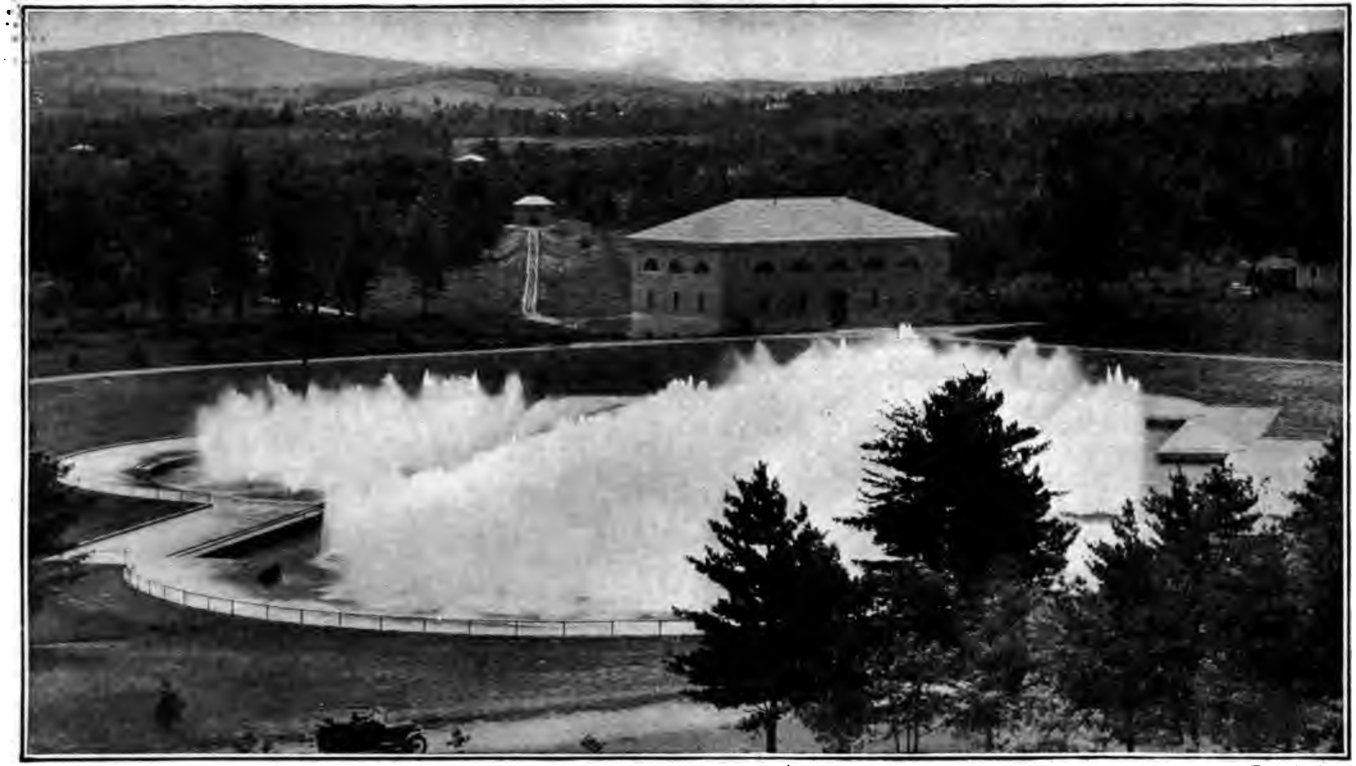

Frontispiece

Ashokan Aërator, near Inlet of Catskill Aqueduct, Discharging 500,000,000 gals. per day through 1558 Nozzles, most of them $1\frac{3}{8}$ in. in Diameter.

CONVEYANCE AND DISTRIBUTION OF WATER FOR WATER SUPPLY

AQUEDUCTS, PIPE-LINES AND DISTRIBUTING SYSTEMS

A PRACTICAL TREATISE FOR WATER-WORKS ENGINEERS AND SUPERINTENDENTS

BY

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and " The Design and Construction of Dams"

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PREFACE

During the past fifty years, great progress has been made in the construction and operation of water works. Large aqueducts and reservoirs have been built, the efficiency of pumping engines has been improved, and the purification of water has been greatly advanced.

A number of good treatises on water supply have been published during recent years, but as they all endeavor to cover the whole subject, they necessarily fail to discuss some parts in sufficient detail to be of practical value to the water works engineer and superintendent.

A complete treatise on water supply, including the collection and storage of water, its conveyance and distribution, pumping engines, water purification, and the maintenance and operation of water works, would require several volumes. Some of these subjects have already been fully discussed in separate treatises, but very little has thus far been published about the distribution of water, the detection and prevention of waste of water by means of Pitot tube gaugings and water meters; about fire protection, high-pressure water systems, tapping machines, valve-inserting machines, water stage recorders, and the many other devices and appurtenances with which the water works engineer and superintendent should be familiar.

The author has endeavored to cover these subjects in the present volume. The facts given were obtained by practical experience, observation, study and correspondence during a service of over thirty years as one of the engineers of the water works of the City of New York; first, on the construction of the New Croton Aqueduct and Reservoirs, and later as consulting engineer of the Department of Water Supply, Gas and Electricity of said city, which has charge of the maintenance and operation of the City's water works.

In discussing the various appurtenances and devices used on water works — such as gates, valves, hydrants, water meters, water-stage recorders, etc., — the author has described the leading types and makes. It is hoped that this feature, which makes the book somewhat encyclopedic, will save the water works engineer and superintendent much time and correspondence when he has occasion to consider the merits of different makes of appurtenances and devices.

The chapters on submerged pipes, intake-pipes and tunnels describe a number of different methods that have been adopted for laying pipes under water. This information, which is given for the benefit of engineers who have not access to large technical libraries, has been condensed from articles contained in technical papers and in the Transactions of Engineering Societies.

The descriptions of aqueducts given in Chapter XV were submitted to the engineers in charge of the different works, in order to insure accuracy. For the same reason, the chapters on the different kinds of pipes, appurtenances, and devices were sent to the manufacturers of these articles to enable them to correct errors and to make suggestions. The chapters on Pitot tube gaugings, water meters, fire protection, etc., were thoroughly discussed with experts on these subjects.

It would make a long list if the author were to mention here all the many engineers, superintendents, and manufacturers who have kindly assisted him in the preparation of this book by furnishing information or by examining certain parts of his text. To all who have assisted him in this manner the author wishes to express here his most hearty thanks.

It has been the author's aim in all cases to state the sources of his information and to give full credit where it was due. If he has failed to do so, in any particular case, he will be glad to make proper acknowledgment in a future edition, if such should be required, providing that his attention is drawn to the omission in giving proper credit.

The present volume deals only with the conveyance and distribution of water for water supply. If it is found to be of value to water works men, the author hopes to have a volume, which he is now preparing on "The Collection and Storage of Water," published later.

E. W.

June 1, 1918.

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PART I

WATER CONSUMPTION AND HYDRAULIC FORMULAS

CONVEYANCE AND DISTRIBUTION OF WATER

CHAPTER I

1. Consumption of Water. — All plans for water works for a community are based upon the existing, and probably future, consumption of water. The quantity of water consumed daily per capita varies greatly in different countries, and in different communities of the same country. In Europe this quantity is much smaller than in America, largely owing to the fact that modern improvements have not been introduced to the same extent in the former as in the latter country.

A great deal of water is unintentionally or wilfully wasted in American cities. Large quantities are, also, used for industrial purposes. Unless the water consumed is subdivided, according to the different purposes for which it is used, no satisfactory comparison can be made between the water consumptions of different cities.

Several associations of water works engineers and superintendents have collected statistics of water consumption, subdivided as much as possible.* As soon as some uniform system of making reports of the operation of water works has been adopted, it will be possible to make valuable deductions. The Committee of the American Water Works Association on Water Consumption has recommended that water consumption be subdivided as follows:

1. Domestic consumption, including what is used wholly for residential or domestic purposes.
2. Non-domestic consumption:
   - (*a*) Industrial use for railroad, factories, etc.
   - (*b*) Commercial use for stores, saloons, stables, boarding houses, etc., including double houses.
   - (*c*) Public use for fire protection, street sprinkling, sewer flushing, public buildings, schools, fountains, etc.

Industrial use varies greatly in different communities, but the daily per capita consumption for commercial purposes is more uniform. The

* Jour. New England Water Works Assoc., 1913, XXVII, p. 29; Proc. Amer. Water Works Assoc., 1915, p. 181; Report on "Twenty-one Statistical Summaries of the Results of Operating Water Works," made in 1910 by a committee of the "Deutscher Verein von Gas und Wasserfachmännern."

quantity of water used for public purposes depends largely upon whether this water is free, in consideration of franchise rights, etc., or not. In municipal water works, the keeping of a record of what is used for public purposes tends to reduce the quantity.

From the data which the above mentioned committee has collected in 1913 to 1915, Table I, showing the subdivision of water consumption

TABLE I. — WATER CONSUMPTION IN AMERICAN CITIES IN 1914
(Compiled from data given in Proc. Amer. Water Works Assoc., 1915, pp. 183–200.)

City	Population	Average daily consumption		Percentages of total average daily consumption per capita			
		Total	Per capita	Domestic consumption and water not accounted for	Registered non-domestic consumption		
					Industrial	Commercial	Public
Auburn, N. Y.	36,363	6,550,000	180.0	78.8	11.2	7.0	4.0
Biddeford and Saco, Me.	68,000	1,736,000	60.0	80.7	16.1	2.5	0.7
Corning, N. Y.	14,900	1,240,000	83.3	69.2	20.2	8.8	1.8
Clinton, Ia.	25,500	1,570,000	61.4	77.9	10.0	7.6	4.5
Edgeworth Water Co.	3,358	358,000	106.6	77.7	20.8	0.3	1.2
Elyria, Ohio	16,000	1,938,000	121.1	50.6	38.8	6.0	4.6
Fort Smith, Ark.	28,000	2,531,000	90.4	72.5	5.8	19.0	2.7
Holyoke, Mass.	60,000	6,780,000	113.0	72.3	14.2	10.2	3.3
Jefferson City, Mo.	13,500	1,335,000	99.0	38.2	53.7	4.1	4.0
Jersey City, N. J.	300,000	44,000,000	146.7	67.6	29.6	0.1	2.7
Kokomo, Ind.	20,000	1,950,000	97.5	84.6	11.5	2.6	1.3
Keokuk, Ia.	14,000	1,468,000	105.0	81.9	10.0	7.0	1.1
Louisiana, Mo.	4,860	475,000	97.7	76.5	20.1	1.7	1.7
Lexington, Ky.	40,000	2,483,700	62.1	31.8	28.8	25.8	13.6
Madison, Wis.	27,000	2,127,000	79.0	80.5	7.4	7.9	4.2
Marinette, Wis.	14,610	1,363,270	93.3	85.9	6.2	2.6	0.3
Milwaukee, Wis.	430,000	47,920,000	111.4	27.2	38.2	29.4	5.2
Mount Vernon, Ind.	5,800	735,616	126.8	85.2	11.5	1.2	2.1
New Castle, Pa.	36,000	3,536,000	98.2	69.1	23.7	6.4	0.8
New Orleans, La.	360,000	20,600,000	57.2	72.0	*	22.4	5.6
Oak Park, Ill.	26,000	1,790,000	69.0	81.7	12.4	2.3	3.6
Portsmouth, Va.	75,000	4,867,486	64.9	65.0	30.6	2.5	1.9
Quincy, Ill.	38,587	1,923,300	50.0	52.7	32.4	10.9	4.0
Racine, Wis.	41,000	3,444,900	84.0	56.4	31.8	9.2	2.6
Richmond, Va.	120,000	13,861,245	115.5	41.4	27.9	22.0	8.7
Richmond, Ind.	23,000	2,615,175	113.7	49.1	34.6	6.9	9.4
Ripan, Wis.	3,800	289,700	76.2	88.0	5.7	4.3	2.0
Rochester, N. Y.	250,000	23,740,000	95.0	62.7	19.3	12.9	5.1
Scottdale, Pa.	8,500	2,074,000	244.0	76.2	23.0	0.5	0.3
St. Joseph, Mo.	89,403	9,000,000	101.0	48.3	39.9	8.0	3.8
St. Louis, Mo.	730,000	82,100,000	101.0	70.5	17.4	10.8	1.3
San Diego, Cal.	85,000	6,853,863	80.6	62.5	9.0	14.9	13.6
Spokane, Wash.	104,402	29,869,000	286.1	93.0	1.4	0.6	5.0
Washington, D. C.	353,000	57,500,000	162.7	76.0	10.9	11.5	1.6
Wichita, Kan.	54,545	3,820,000	70.0	83.8	3.1	11.2	1.9

* Included in commercial consumption.

TABLE II. — WATER CONSUMPTION IN AMERICAN CITIES IN 1912
(Compiled from data given in Journal New England Water Works Association 1913, XXVII, pp. 34 and 35.)

City	Population served	Taps		Water consumption			Meter rates per 100 cubic feet	
		Number live	Per cent metered	Average per capita per day, gal.	Maximum in percentage of average per capita for		Max., cents	Min., cents
					1 month	1 day		
Ansonia, Conn.	10,000	1,010	16	80	....	....	18	5
Arlington, Mass.	10,500	2,065	62	84	....	....	15	11
Baltimore, Md.	589,000	110,395	30	121	116	116	....	4.5
Bangor, Me.	24,803	4,850	20	161	....	....	55	....
Bath, Me.	9,396	2,328	....	185	109	119	25	8
Calais, Me.	7,500	1,428	....	94	106	....	25	8
Boston, Mass.	670,585	......	....	....	....	....	....	....
Brewer, Me.	6,500	1,325	....	....	....	....	....	....
Cambridge, Mass.	104,829	15,905	30	101	....	....	15	7.5
Canandaigua, N. Y.	7,500	1,546	110	100	127	154	6	6
Claremont, N. H.	4,500	1,062	....	....	....	....	15	$7\frac{1}{2}$
Davenport, Ia.	43,000	7,500	67	93	111	147	....	....
Cleveland, O.	618,000	83,011	98	....	....	....	....	....
Denison, Tex.	11,000	2,125	....	56	....	192	50	15
Dover, N. H.	12,500	1,901	....	53	....	....	$22\frac{1}{2}$	15
Fall River, Mass.	118,645	8,501	99	44	104	124	....	....
Fitchburg, Mass.	34,000	5,200	71	....	....	....	....	....
Gardner, Mass.	13,965	1,732	7	46	....	....	....	....
Hartford, Conn.	121,644	12,621	98	66	....	....	....	....
Holyoke, Mass.	56,901	4,280	8	103	....	....	....	....
Lawrence, Mass.	84,000	7,641	98	46	....	....	....	....
Hyde Park, Mass.	15,506	2,521	48	74	....	....	26	18.2
Jamestown, N. Y.	32,000	6,504	51	99	....	....	20	6
Johnstown, Pa.	55,000	10,397	4	157	....	125	27	5
Kingston, Mass.	2,000	475	....	120	....	137	....	....
Lexington, Mass.	5,000	838	57	69	....	....	....	....
Lowell, Mass.	106,294	12,494	79	51	....	....	14	10
Manchester, N. H.	73,000	6,700	77	52	....	....	$12\frac{1}{2}$	8
Middleboro, Mass.	8,230	1,000	54	41	....	....	$18\frac{3}{4}$	$7\frac{1}{2}$
Milwaukee, Wis.	400,000	56,490	98	112	....	....	$18\frac{3}{4}$	$7\frac{1}{2}$
Nantucket, Mass.	2,500	1,160	0	72	....	....	....	....
Nashua, N. H.	25,000	3,759	25	94	105	159	15	$3\frac{3}{4}$
New London, Conn.	19,000	4,132	17	140	....	....	12	$4\frac{1}{2}$
Newport, R. I.	25,025	5,522	3	157	....	....	....	....
New Rochelle, N. Y.	37,500	5,810	99	85	....	....	15	....
Pawtucket, R. I.	78,919	10,602	86	80	131	170	22.4	4.5
Peoria, Ill.	67,000	9,170	4	119	119	154	15	$4\frac{1}{2}$
Plymouth, Mass.	11,000	2,480	4	114	150	178	....	....
Providence, R. I.	246,000	27,818	89	63	109	137	15	$7\frac{1}{2}$
Reading, Mass.	5,500	1,355	....	37	....	....	....	....
Saginaw, Mich.	51,510	5,871	6	186	....	....	....	....
Somerville, Mass.	80,000	12,357	53	74	....	....	12	8
Springfield, Mass.	87,500	12,545	61	121	....	....	22	5
Washington, D. C.	331,000	65,215	29	182	107	150	3	....
Watertown, Mass.	12,800	2,042	100	68	....	....	25	5
Wilkinsburg, Pa.	86,755	11,788	....	90	....	....	20	10
Wilmington, Del.	87,500	18,414	35	118	....	....	7.5	36
Windsor, Conn.	1,200	248	10	57	....	....	24	20
Woonsocket, R. I.	44,200	3,309	96	....	117	143	22.5	....
Worcester, Mass.	156,362	......	....	66	....	....	15	7.5

TABLE III. — WATER CONSUMPTION IN ENGLISH CITIES IN 1913–14

(Compiled from data given in " Water Works Directory and Statistics," 1913–14, London.)

City	Population supplied 1912	Average daily consumption per capita				Price of water	
		In U. S. gallons		In Imperial gallons		Per 1000 U. S. gallons	Per 1000 Imperial gallons
		Domestic	Trade, etc.	Domestic	Trade, etc.	Cents	Shillings or Pence
Aberdeen *a*	166,700	32.40	16.80	27.00	14.00	12–3	7–2
Ashton-under-Lyne	147,235	20.10	4.65	16.75	3.88	....	....
Belfast	400,000	29.81	17.37	24.84	14.48	17–9	10–5½
Birmingham	851,845	20.02	12.22	16.68	10.18	....	....
Blackburn	143,000	17.35	9.88	14.46	8.23	15	8.9
Bolton	295,480	......	......	......	......	15	9
Bournemouth	90,000	39.00		32.5		40 max.	2*s*. = max.
Bradford	450,000	27.00	25.80	22.50	21.50	15–13	9–7.5
Bristol	372,110	32.81		27.34		30–12	18–7
Cardiff	227,000	21.11	13.43	17.59	11.19	30–10	18–6
Darlington *a*	57,000	25.20	......	21.00	......	27	16
Derby	133,000	21.36	11.35	17.80	9.46	20–10	12–6
Dublin	380,000	27.89	16.51	23.24	13.76	10	6
Dundee	210,000	60.00		50.00		13–12	8–7
Edinburgh and Leith	450,000	41.64	14.40	34.70	12.00	10	6
Glasgow	1,132,025	48.60	26.40	40.50	22.00	5–1	3¾
Halifax	231,000	18.00	15.60	15.00	13.00	13–10	8–6
Hastings	61,000	21.60		18.00		40	24
Huddersfield	156,420	20.40	12.00	17.00	10.00	15–13	9–7½
Hull	290,875	35.52	12.07	29.60	10.06	10	6 Trade
Leeds	483,230	28.67	13.16	23.89	10.97	17–12	10–7
Leicester	278,217	18.14	7.34	15.12	6.12	20–10	12–6
Liverpool	944,963	26.36	14.53	21.13	12.11*b*	10	6 Trade
London	7,130,420	40.82		34.02		....	....
Manchester	1,200,000	24.00	18.00	20.00	15.00	15–5	9–3
Newcastle on Tyne	585,000	22.16	17.40	18.47	14.50	23–9	1*s*. 2*d*.–5½*d*.
Northampton	120,000	15.24	3.72	12.7	3.10	30	1*s*. 6*d*.
Nottingham	372,978	13.37	12.67	11.14	10.56	15	9*d*.
Oldham	233,000	21.60	11.40	18.00	9.50	40–20	2–1*s*. Trade
Paisley	116,000	48.60	44.10	40.50	36.75	15–2	9–1
Plymouth	143,553	39.16	14.65	32.63	12.21	10–6	6–3½
Portsmouth	251,492	28.22		23.52*c*		30–13	1*s*. 6*d*.–8*d*.
Rochdale	130,000	22.49	3.53	18.74	2.96	....	....
Sheffield	701,141	......	......	......	......	17–10	10*d*.–6*d*.
South Essex	197,000	......	......	......	......	17–13	10 *d*.–8*d*.
Stockport	174,000	23.40	6.30	19.50	5.25	40–15	2*s*.–9*d*.
Stockton	260,000	19.88	44.40	16.57	37.00	27–8	1*s*. 4*d*.–4¼*d*.
Swansea	120,000	......	......	......	......	23–1	1*s*. 2¾*d*.–4*d*.
Wakefield	160,000	......	......	......	......	15–12	9*d*.–7*d*.
Wolverhampton	161,000	20.04	7.56	16.70	6.30	27–10	1*s*. 4*d*.–6*d*.

a All supplies include neighboring communities except those marked.
b In city only.
c Domestic and unmetered trade.

TABLE IV. — WATER CONSUMPTION IN FRENCH CITIES *

(Compiled from data given by Debauve and Imbeaux in " Distribution d'Eau," Vol. II, pp. 11 and 12, Paris, 1905.)

City	Population, 1901	Average daily consumption, per capita		Private service. Number of taps	Price of water in 1902	
		U. S. gals.	Liters		Per 1000 U. S. gals. in cents	Per cubic meter in centimes
Albi	22,571	58	220	1,275	14	17.2
Amiens	90,758	24	91	6,600	22	30
Angers	82,398	38	144	5,000	13	18
Angoulême	37,650	35	134	1,390	10	14
Arcachon	8,259	233	882	950	22	30
Auxerre	18,901	50	288	1,200	17	23.4
Avignon	46,786	36	138	2,203	8	10.9
Bar-le-Due	17,693	31	118	840	20	28
Beaune	13,887	54	206	446	11	15
Beçancon	55,362	131	495	1,707	14	19
Bordeaux	256,638	46	175	22,129	16	22
Boulogne	49,940	38	145	1,698	12	16.4
Bourg	18,887	74	280	678	11	15
Bourges	46,551	28	106	2,300	12	16.4
Brest	84,184	7	27	2,350	45	61.6
Caen	44,794	51	197	1,379	18	24
Calais	59,743	75	282	6,226	15–44	20–60
Cannes	30,420	225	852	3,500	2–4	2.5–5.6
Chartres	23,431	27	101	1,300	15	20
Chateauroux	24,957	25	94	1,000	18	25
Cherbourg	42,938	32	121	2,011	24	33
For domestic purposes only						
Dieppe	22,839	143	540	1,054	10	13
Digne	7,238	74	281	1,054	18	25
Dijon	71,326	50	190	2,000	18	25
Dunkerque	38,925	24	89	2,000	36	49.5
Epinal	28,080	69	261	1,030	11	15
Foix	7,065	64	242	118	11	15
Gap	11,018	30	112	10	4	5
Grenoble	68,615	272	1,032	7,031	4	5.5
Langres	9,921	19	73	238	30	41
La Rochelle	31,559	....	156	2,650	18	25
Le Havre	130,196	41	119	7,141	28	38.3
Le Mans	63,227	47	179	3,442	11	15
Limoges	84,121	30	113	2,450	16	21.9
Lorient	44,640	32	120	695	14	19
Lyon	459,099	50	188	51,141	16	22
Macon	18,928	50	189	3,100	22	30
Mont-de-Marsan	11,604	15	56	950	15	20
Montelimar	13,351	155	585	820	8	11.5
Montpelier	75,950	88	332	2,850	22	30
Nevers	27,673	21	80	1,200	22	30
Nîsmes	80,605	47	179	2,395	15	20
Orleans	67,311	31	116	5,068	20	27
Pau	34,268	81	307	14,200	10	14
Pernigneux	31,976	56	213	1,935	5–7	7–10
Perpignan	36,157	125	472	1,026	16	22
Portiers	39,886	24	92	2,330	14	18.8
Reims	108,385	52	198	5,000	20	27
Rennes	74,676	52	196	2,192	20	27.4
Rochefort	36,458	24	92	1,626	18	24.7
Saint-Brieuc	22,198	25	96	400	18	24.3
Saint-Etienne	146,559	55	207	6,500	20	27.4
Tarbes	26,055	61	259	410	12	16.4
Toul	12,287	17	66	170	7	10
Toulon	101,602	40	152	3,633	12	16.4
Toulouse	149,841	69	265	5,000	18	25
Tours	64,695	54	205	4,350	9	12
Trouville	6,137	39	146	500	37	50
Troyes	53,146	78	294	1,150	13	18
Valence	29,946	104	394	1,000	8	11.6
Versailles	54,983	67	253	2,975	20	27.4
Vichy	14,254	63	240	2,975	3	4

* Some large cities, like Paris and Marseilles, are omitted in this table as they have a double system of water supply: one for domestic purposes and the other for public service.

TABLE V. — WATER CONSUMPTION IN GERMAN CITIES IN 1909–11

(Compiled from data in Otto Lueger's "Wasserversorgung der Städte," Vol. I, second edition, pp. 126–129.)

City	Water meters *	Population, 1910	Average daily consumption per capita		Classification of use in percentage of average consumption			
			Gals.	Liters	Domestic	Industrial	Public	Waste
Ludwiglust	*a*	7,000	3	11	......	......	......	......
Klingenthal	*a*	6,000	4	14	......	......	......	......
Hildesheim	*b*	49,000	10	37	......	......	......	......
Plauen	*b*	114,000	10	39	6.0	6.4	82.7	4.9
Hof	*b*	41,000	11	40	......	......	......	......
Guben	*b*	38,000	13	49	......	......	......	......
Chemnitz	*b*	275,000	13	51	25.3	74.7		......
Mainz	*a*	115,000	16	60	7.0	6.3	76.8	9.9
Spandau	*b*	84,000	16	60	17.0	5.1	72.5	5.4
Stettin	*b*	241,000	16	61	4.7	8.4	68.3	18.6
Offenbach	*a*	72,000	17	64	9.5	9.8	71.2	9.5
Kiel	*a*	185,000	17	66	8.8	1.8	66.9	22.5
Leipzig	*b*	579,000	......	68	5.9	82.0		12.1
Gleiwitz	*a*	68,000	19	71	37.4	3.7	50.2	8.7
Zwickau	*b*	75,000	19	71	2.6	41.5	55.2	0.7
Halle	*b*	188,000	20	76	2.2	5.4	71.2	21.2
Köningsberg	*b*	241,000	......	77	5.1	65.2		29.7
Braunschweig	*a*	145,000	......	78	12.3	80.7		7.0
Bielefeld	*b*	78,000	21	81	3.0	3.8	77.3	15.9
Bonn	*b*	91,000	......	81	7.6		69.0	23.5
Berlin	*b*	2,200,000	22	82	6.7	84.0		9.3
Ludwigsburg	*b*	23,000	......	82	......	......	......	......
Nürnberg	*b*	320,000	22	82	13.8	7.9	73.8	4.5
Kassel	*b*	158,000	22	83	19.5	51.3		29.2
Breslau	*b*	505,000	22	84	12.6	8.5	72.3	6.6
Danzig	*b*	160,000	23	87	10.0	3.0	57.3	29.7
Posen	*b*	154,000	23	88	3.2	4.9	43.5	48.4
Remscheid	*b*	61,000	24	89	14.4	73.1		12.5
Mannheim	*b*	184,000	24	92	15.5	6.9	64.8	12.8
Magdeburg	*b*	251,000	25	93	4.3	7.0	74.3	14.4
Aachen	*b*	154,000	25	95	0.9	4.6	69.8	24.7
Darmstadt	*a*	87,000	25	95	4.3	8.5	79.8	7.4
Charlottenburg	*b*	764,000	25	96	4.2	2.9	75.9	17.0
Dresden	*b*	540,000	26	98	8.3	74.3		17.4
Stuttgart	*d*	275,000	27	101	10.7	1.6	78.1	9.6
Karlsruhe	*c*	129,000	30	113	11.1	10.7	76.0	2.2
Wiesbaden	*a*	113,000	31	116	6.6	8.0	66.1	19.3
Muhlhausen, I. E.	*b*	119,000	31	120	23.8	8.0	68.2	......
Mühlheim a. R.	*a*	138,000	32	122	8.3	0.8	83.5	7.4
Strassburg	*b*	176,000	33	124	50.3	13.0	35.6	1.1
Hanover	*b*	274,000	34	128	21.7		69.8	8.5
Koblenz	*c*	57,000	34	130	39.2	3.3	52.4	5.1
Rostock	*b*	70,000	35	131	4.6	94.4		1.0
Essen (Stadt)	*a*	297,000	35	133	0.6	5.9	68.3	25.2
Cologne	*b*	450,000	36	136	21.3	66.6		12.1
Krefeld	*d*	128,000	36	138	3.1	8.3	87.6	1.0
Düsseldorf	*b*	348,000	37	139	8.1	80.0		11.9
Elberfeld	*a*	205,000	37	139	0.9	2.4	70.7	26.0
Hamburg	*a*	888,000	37	141	2.6	96.5		0.9
Altona-Blankenese	*d*	191,000	38	144	1.9	3.6	94.5	......
Barmen	*b*	170,000	42	158	0.7	1.8	64.8	32.7
Hagen i. W.	*b*	86,000	44	165	0.3	88.4		11.3
Bremen	*d*	242,000	44	168	......	......	......	......
Ulm	*d*	56,000	45	171	......	......	......	......
Cleve	*b*	19,000	54	206	......	......	......	......
Würzburg	*d*	85,000	55	209	26.6	4.4	52.7	16.3
München	*b*	571,000	57	215	23.6	7.7	68.6	0.1
Dortmund	*b*	340,000	64	244	19.6	80.4		......
Augsburg	......	100,000	67	255	7.0		92.7	0.3
Freiburg i. B.	*b*	81,000	78	297	12.1	5.2	55.5	27.2

* *a* Practically all *services* metered.
b *Services* generally metered.
c *Services* generally not metered
d Practically no *services* metered

TABLE VI. — WATER CONSUMPTION IN VARIOUS FOREIGN CITIES

(Compiled from data given in "Die Wasserversorgung der Städte," by Otto Lueger, Vol. I, 2d edition, 1914, p. 130, and from "Water Works Handbook," by A. D. Flinn, R. S. Weston, and C. L. Bogert, New York 1916, p. 549.)

City	Country	Year	Population	Average daily consumption per capita	
				In U. S. gallons	In liters
Petrograd	Russia	1913	2,018,000	38	144
Moscow	"	1910	1,335,000	14	53
Odessa	"	1912	580,000	19	72
Riga	"	1910	283,000	20	77
Warsaw	"	1912	771,000	25	95
Stockholm	Sweden	1912	342,000	26	97
Christiania	Norway	1912	244,000	33	124
Amsterdam	Holland	1911	566,000	26	98
Rotterdam	"	1912	441,000	29	110
Copenhagen	Denmark	1913	476,000	33	125
Vienna	Austria	1911	2,065,000	25	95
Krakow	"	1910	152,000	8	31
Pressburg	"	1910	78,000	16	60
Budapest	"	1913	910,000	58	220
Salzburg	"	1910	36,000	48	183
Brussels	Belgium	1910	312,000	25	95
Zurich	Switzerland	1910	191,000	58	220
Geneva	"	1913	131,000	217	821
Basel	"	1910	132,000	42	158
Winterthur	"	1910	25,000	185	520
Madrid	Spain	1913	570,000	84	318
Rome	Italy	1911	542,000	120	454
Venice	"	1911	132,000	40	151
Florence	"	1908	233,000	28	107
Milan	"	1908	599,000	27	101
Leghorn	"	1908	105,000	16	59
Trapani	"	1908	59,000	48	181
Taranto	"	1908	61,000	15	55
Athens	Greece	1913	188,000	34	129
Alexandria	Egypt	1912	420,000	28	106
Cairo	"	1912	705,000	25	95
Bombay	India	1913	979,000	37	140
Calcutta	"	1913	1,109,000	62	235
Montreal	Canada	1912	600,000	153	579
Toronto	"	1911	425,000	118	447
Vancouver	"	1912	125,000	173	416
Winnipeg	"	1912	225,000	56	212
Rio Janeiro	Brazil	1913	1,000,000	60	227
Sao Paulo	"	1913	332,000	63	238
Buenos Aires	Argentine	1913	1,252,000	35	132
Montevideo	Uruguay	1913	363,000	11	42
Sidney	Australia	1912	731,000	50	189
Tokio	Japan	1912	1,447,000	32	121

in a number of communities, has been compiled. Table II, taken from data collected by a Committee of the New England Water Works Association, gives the daily consumption per capita in different cities and towns in the United States, without subdivisions. The daily per capita consumption in English, French, German, and other European cities, and in various parts of the world, is given, respectively, in Tables III to VI.

2. Variations in Rate of Consumption. — In the tables mentioned above, only the average daily consumption per capita has been given. The use of water varies, however, according to the season, the day of the week, and the hour of the day or night (Figs. 1 and 2).* There is, therefore, considerable variation in the rate at which water is con-

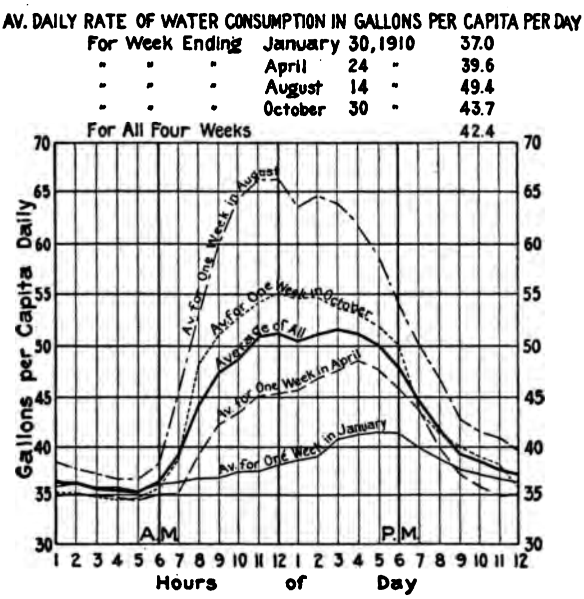


FIG. 1.— Hourly Variations in Water Consumption in Fall River, Mass. (Population, 119,295 in 1910.)

sumed, and, in designing water works, allowance must be made for what may be the maximum monthly, weekly, daily and hourly rate of consumption. These fluctuations may be taken care of to a certain extent, by the construction of a distributing reservoir of sufficient capacity — usually storing three to ten days' supply — or, in the direct system, by a reserve pumping capacity, but the distributing pipes should be made large enough to deliver about the maximum daily draft and, at the same time, as much water as may be needed for extinguishing fires.

* Jour. New England Water Works Assoc., 1913, XXVII, pp. 90 and 92.

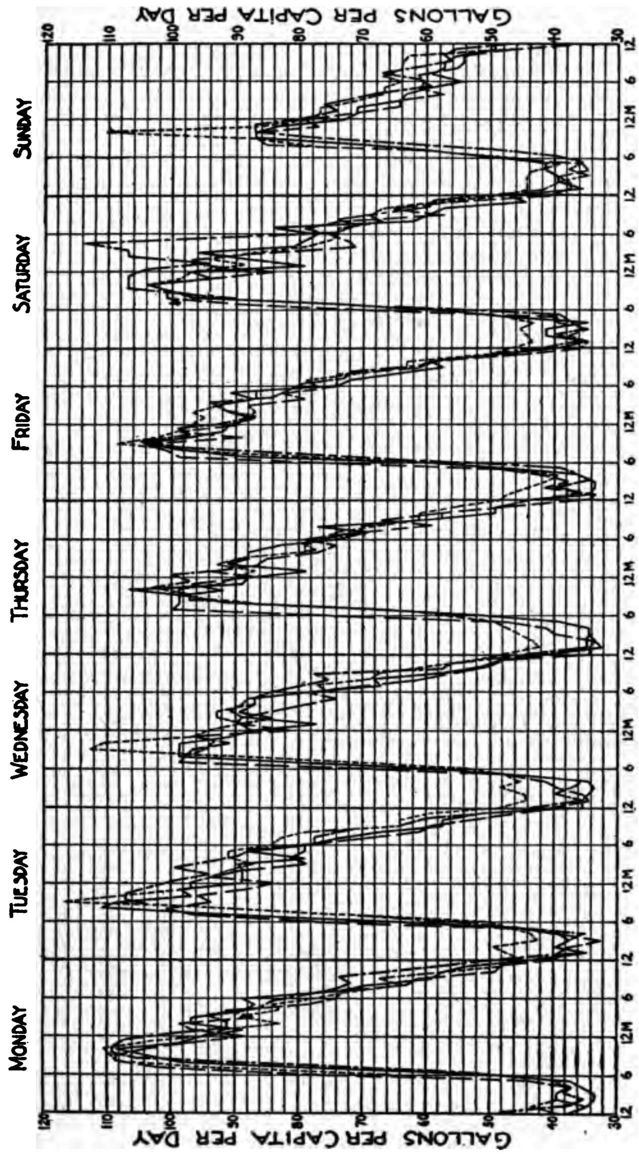


FIG. 2. — Hourly Variations in Water Consumption in Hartford, Conn., During Four Typical Weeks in 1911. (Population, 98,915.)

TABLE VII.* — MAXIMUM WATER CONSUMPTION FOR MASSACHUSETTS CITIES AND TOWNS, 1910

City or town	Population	Average daily consumption per capita	Maximum consumption in per cent of average for the year		
			Monthly	Weekly	Daily
Worcester	145,986	74	115	....	139
Fall River	119,295	44	107	114	123
Lowell	106,294	51	112	129	147
Cambridge	104,839	100	106	111	119
Lynn and Saugus	97,383	72	110	121	150
New Bedford	96,652	81	109	121	131
Lawrence	85,892	45	113	133	133
Brockton	56,878	39	115	141	177
Salem	43,697	90	112	114	148
Newton	39,806	63	118	130	150
Taunton	34,259	63	111	118	138
Waltham	27,834	88	108	111	123
Brookline	27,792	89	116	132	177
Gloucester	24,398	55	156	207	237
Beverly	18,650	91	160	210	246
Attleborough	16,215	54	115	116	169
Peabody	15,721	168	118	108	161
Woburn	15,308	139	124	137	167
Milford and Hopedale	15,243	51	118	125	139
Newburyport	14,949	68	122	138	178
Gardner	14,699	44	111	125	246
Marlborough	14,579	37	114	159	190
Framingham	12,948	48	121	137	179
Abington and Rockland	12,383	45	137	151	197
Plymouth	12,141	103	127	136	166
Webster	11,509	38	129	139	189
Methuen	11,448	38	142	176	182
Wakefield	11,404	61	139	175	208
Bridgewater and East Bridgewater	11,051	22	123	132	177
Danvers and Middleton	10,536	89	121	153	178
Amesbury	9,894	44	114	116	155
Natick	9,866	57	123	144	298
North Attleborough	9,562	52	140	158	183
Dedham	9,284	129	119	128	141
Middleborough	8,214	42	126	155	214
Braintree	8,066	81	107	115	133
Montague and Erving	8,014	66	114	106	232
Norwood	8,014	63	136	136	211
Marblehead	7,338	79	186	214	237
Andover	7,301	86	115	....	189
Whitman	7,292	29	145	152	....
Randolph and Holbrook	7,117	74	162	189	237
Hudson	6,743	49	123	133	....
Maynard	6,390	36	108	130	167
Stoughton	6,316	35	123	166	214
Reading	5,818	35	149	172	189
Ipswich	5,777	42	143	200	253
Grafton	5,705	18	122	133	178
Winchendon	5,678	30	117	133	150
Franklin	5,641	61	144	158	288
North Andover	5,529	40	132	160	195

* This table is based upon data furnished by X. H. Goodnough, Chief Engineer, Massachusetts State Board of Health, to the New England Water Works Assoc. (Jour. N. E. W. W. Assoc., 1913, XXVII, p. 98).

TABLE VII. — *Continued*

Wellesley..........................	5,413	61	111	129	184
Orange............................	5,282	26	131	158	182
Mansfield..........................	5,183	75	129	137	368
Easton............................	5,139	24	117	158	263
Needham..........................	5,026	66	133	148	180
Walpole...........................	4,892	102	117	146	247
Canton............................	4,797	61	123	138	159
Provincetown......................	4,369	38	182	203	245
Rockport..........................	4,211	72	205	272	295
Foxborough........................	3,863	50	102	118	150
North Brookfield...................	3,075	66	123	170	319
Nantucket.........................	2,962	67	191	230	263
Ayer..............................	2,797	50	128	144	342
Manchester........................	2,673	120	217	271	302
Sharon............................	2,310	57	170	210	240
Avon..............................	2,013	36	153	200	283

TABLE VIII. — HOURLY FLUCTUATIONS IN RATE OF WATER CONSUMPTION, IN GALLONS PER CAPITA

Locality.....	Holyoke, Mass.	Springfield Mass.	Fall River, Mass.	Hartford, Conn.	Woonsocket R. I.	Peoria, Ill.
Population..	51,000	87,500	119,295	121,644	38,125	66,950
Week ending.	Nov. 17, 1905	Summer, 1910	Aug. 14, 1910	Aug. 7, 1911	Aug. 19, 1911	July 7, 1910
Time	Gallons	Gallons	Gallons	Gallons	Gallons	Gallons
1 A.M.	68	71	38	37	19	110
2	69	71	37	38	17	225
3	67	86	36	37	18	170
4	68	83	34	38	17	180
5	68	103	34	43	19	223
6	80	130	37	56	30	228
7	98	157	45	77	50	245
8	117	176	56	95	58	340
9	123	179	60	100	59	340
10	127	178	64	100	53	330
11	125	168	67	92	53	335
12	127	161	63	88	52	323
1 P.M.	123	173	64	82	49	330
2	125	172	65	88	54	350
3	123	170	63	89	51	320
4	117	153	62	82	48	335
5	109	140	58	82	48	330
6	105	127	54	75	49	428
7	97	118	50	71	43	426
8	97	107	47	67	38	451
9	90	94	43	59	30	300
10	85	87	42	56	26	220
11	77	80	41	50	24	268
12	74	75	40	43	20	205
Rate of consumption						
Average.....	98	127	50	60	38	298
Maximum....	127	179	67	100	59	451
[illegible]m....	67	71	34	37	17	110
	129	141	134	167	155	155

Table VII gives the average daily consumption of water in Massachusetts cities; and, also, the maximum monthly, weekly, and daily consumption, in per cent of the average for the year. According to this table, the maximum increase from the average daily per capita consumption of the year may be as follows: average daily, 100; maximum monthly, 100 to 217; maximum weekly, 111 to 272; maximum daily, 123 to 368.

Table VIII shows the hourly fluctuations in water consumption in six American cities.* According to this table the maximum hourly rate of consumption may be 167 per cent of the average hourly rate. If such a fluctuation should occur on the day of maximum consumption in the year, which is not probable, it might make, according to Tables VII and VIII, the maximum rate of consumption 615 per cent of the average rate.

In actual practice, provision for hourly fluctuation in consumption is made, as already stated, by storage capacity in a reservoir or standpipe; or, in the direct system, by a reserve pumping capacity. The only effect of the hourly variations will be, in ordinary cases, to reduce the available pressure in the water mains.

From the information given above, it appears that the maximum daily consumption may be as much as three times the average daily consumption. In ordinary cases, the increase will, however, be only from one to two times the average consumption. As a general rule, the distributing pipes should be designed to deliver twice the average quantity required per day, when they deliver only water for domestic and industrial consumption. When they have, also, to convey water for extinguishing fires, they should be able to deliver $1\frac{1}{2}$ to 2 times the quantity required for the average daily consumption, in addition to what is needed for extinguishing fires.

3. Water Unaccounted For. — In all water works, including those in which all taps are metered, there is always a difference between the estimated quantities of water supplied and water consumed. This difference, which may vary from 10 to 60 per cent of the total supply, is partly due to mistakes made in the estimated supply — such as erroneous assumptions of the amount of slip in the pumping engines, inaccurate gaugings of the supply, under-registry of the water meter, etc., — but it is caused, also, largely by leakage from the pipe system and service-pipes. Where meters are not used, leaky house fixtures cause a considerable loss of water. Of late years efforts have been made to reduce this loss in a number of water works by pitometer measurements and by house-inspection.

* Compiled from data given in Jour. New England Water Works Assoc., 1913, XXVII, pp. 86–97.

4. Consumption of Water in the Metropolitan District of Boston, Mass., in 1902. — For a number of years, the officials in charge of the Boston Water Works gathered valuable statistics of the subdivisions of the consumption of water in their city. This work was continued by *The Metropolitan Water and Sewerage Board*, which was given in 1901 charge of the water works and sewer systems of Boston and of seventeen other adjacent cities and towns. The territory of this district includes 142.7 square miles, and had on May 1, 1903, an estimated population of 897,600.

In 1902 the Board ordered the water supplied to each of the cities and towns in the district to be measured, with a view of apportioning the annual assessment among the cities and towns, and, also, to determine any improper use and waste of water, that might exist. Dexter Brackett * has given a full account of the results obtained by these investigations in the Journal of the New England Water Works Association (Vol. XVIII, pp. 107–160).

The distribution of the water was measured by means of 49 Venturi meters, 8 to 48 ins. in diameter, and Mr. Brackett considered the results obtained to be accurate within 2 per cent. The average daily consumption of water in the whole Metropolitan District, during the latter half of 1903, was found to vary from 98,310,800 to 109,080,500 gals., which quantities were $2\frac{1}{2}$ per cent less than the consumption measured by the displacement of the pump plungers at the pumping stations. The average daily consumption per capita for the whole district varied from 108 to 120 gals. In the different cities and towns of the district it varied from 31 to 141 gals.

Investigations were made to determine how much water was used in the different municipalities of the district for domestic, manufacturing, mechanical, trade and public purposes; also, how much water was used unnecessarily, or wasted.

5. *Domestic Use.* — This includes water used for private stables and for watering lawns. Table IX gives the use of water for domestic purposes in four communities, according to water meters, that had only been in use for a few years. The results given in this table agree with the water consumption of other New England towns in 1902.

In tenement houses in the Metropolitan District, the daily per capita consumption of water was found to vary from 13 to 44 gals., as the monthly rentals varied from $12 to $50. The average for all the tenement houses was found to be 29.63 gals. per capita per day.

* Dexter Brackett was the Engineer of the Distribution Department of the Metropolitan Water Works from 1901 to 1907, when he became Chief Engineer of these works.

Mr. Brackett states that, for thirty years preceding the above-mentioned investigations, the number of water fixtures increased in Boston and vicinity much more rapidly than the population, and that it was, therefore, likely that there would be an increase in the use of water for domestic purpose, but he thought that for many years to come this use would not exceed on an average 25 gals. per capita per day.

TABLE IX. — WATER USED FOR DOMESTIC PURPOSES
(According to water meter records.)

City or town	Number of consumers.		Gallons per day		Gallons per day per consumer	
	1901	1902	1901	1902	1901	1902
Belmont	3,600	3,900	63,760	66,630	17.7	17.1
Malden	21,100	22,550	414,030	450,160	19.6	20.0
Milton	6,850	7,450	115,000	143,500	16.8	19.3
Watertown	9,650	10,250	147,200	152,900	15.3	14.8
	41,200	44,150	739,990	813,190	18.0	18.4

6. *Mechanical and Trade Purposes.* — The use of water for these purposes varies greatly in different communities. In 1902 it varied, as nearly as could be ascertained, from about 12 to 46 gals. per capita per day in large American cities. In the Metropolitan District this use of water was found to amount to about 23.5 gals. per capita per day.

7. *Public Purposes.* — This includes water used for public buildings, public fountains, street sprinkling, flushing water pipes and sewers, and for extinguishing fires. In 1902 the quantity of water used for public buildings in the Metropolitan District was found to be as follows:

WATER USED IN PUBLIC BUILDINGS.

	Gallons per capita per day
Schools	0.70
National, state and county buildings, metered	0.58
Hospitals, asylums and jails, metered	0.50
Churches, theaters and clubs, metered	0.40
Public buildings, other than schools, unmetered	1.60
Total for public buildings	3.78

The quantity of water estimated to be used in 1902 for 139 public drinking fountains in the Metropolitan Water District averaged 664,640 gals. per day, equivalent to 0.74 gal. per capita per day, of the population supplied. In most of these fountains water runs continuously.

For sprinkling streets in 1901 in the Metropolitan Water District 700,000,000 gals., equal to 2.13 gals. per capita per day, were used.

It was found difficult to make a close estimate of the quantity of water used for flushing water pipes and sewers, and for extinguishing fires. Although large quantities are used occasionally for these purposes, the total quantity consumed during a year is comparatively small. In the Metropolitan Water District it probably did not exceed, in 1902, 0.20 gal. per capita per day.

Mr. Brackett estimated the quantity of water used in 1902 for the Metropolitan Water District for public purposes, as follows:

WATER USED FOR PUBLIC PURPOSES

	Gallons per capita per day
Public Buildings	3.78
Drinking and ornamental fountains	1.00
Street sprinkling	2.13
Flushing water pipes and sewers and extinguishing fires	0.20
	7.11

From the investigations made in 1902 in the Metropolitan Water District, Mr. Brackett reached the conclusion that the daily legitimate use of water for inhabitants in this district should not have exceeded at that time about 60 gals., subdivided as follows:

	Gallons
Domestic use	25.0
Manufacturing, mechanical and trade use	23.5
Public use	7.0
	55.5

8. *Water Wasted.* — As already stated, the daily quantity of water consumed in the Metropolitan Water District in the latter half of 1902 amounted to 108 to 120 gals. per capita. If only 60 gals. daily per capita were required, according to Mr. Brackett, for domestic, manufacturing, mechanical, trade, and public purposes, it appears that about half the water supplied to this district was wasted by leaky mains, service-pipes, etc.

9. The Water Consumption in the Metropolitan District for Public Purposes.* — Dexter Brackett gives the consumption for public purposes in Boston and certain other towns of the Metropolitan District in 1908 as follows:

* Jour. New England Water Works Assoc., 1913, XXVII, p. 123.

TABLE X. — AVERAGE DAILY WATER CONSUMPTION IN THE METROPOLITAN WATER DISTRICT OF BOSTON, MASS., IN 1908

Boston (population, 643,810)

Consumers	Number	Gallons	Per capita per day
Boston Protective Department Wagon No. 2	....	471,000	0.002
Cemeteries	2	3,186,500	0.013
Colleges	10	51,784,000	0.220
Metropolitan Water and Sewerage Board	....	39,733,800	0.169
National, State, or County Buildings	15	....	....
Navy Yard 211,953,300			
U. S. Gov., Gallups Island 25,783,600			
Post Office 23,621,800			
State Prison 19,799,600			
State House 15,805,200			
Others 34,677,300			
		331,640,800	1.407
Schools:			
Private	14	8,355,200	0.072
Parochial	21	8,512,200	
State Board of Health, Pathological Laboratory	....	351,600	0.010
Street watering, Park Department	....	39,045,600	0.166
		483,080,900	2.050

(Public schools, public buildings owned by the city, street watering, and other public uses were not metered.)

Somerville (population, 74,400)

Consumers	Number	Gallons	Per capita per day
Metropolitan Park Commission, Wellington Bridge	....	162,300	....
Metropolitan Sewerage Works, Alewife Brook Pumping Station	....	1,085,300	......
National, State, or County buildings (armory)	1	543,000	....
St. Joseph's Parochial School	....	225,200	....
Street watering (estimated by street commissioner)	....	63,912,000	....
Tufts College, Metcalf Hall	....	477,200	....
		66,405,000	2.439

(Schools and municipal buildings were not metered.)

Malden (population, 42,140)

Consumers	Number	Gallons	Per capita per day
Cemeteries	2	365,300	....
Fire stations	6	653,800	....
High school field	....	20,000	....
Metropolitan Park boulevard	....	88,100	....
Municipal departments	....	1,142,900	....
National, state, or county buildings	....	1,161,600	....
Schools:			
Public	20	7,004,000	....
Private	1	53,900	....
Parochial	2	214,700	....
Street watering (figured by cartloads)	....	42,468,300	....
		53,172,600	3.448

Medford (population, 21,920)

Consumers	Number	Gallons	Per capita per day
Fire stations	5	393,900	....
Metropolitan Park Commission	....	340,000	....
Metropolitan Water Works, Glenwood Pipe Yard	....	130,000	....
Municipal Departments	....	1,334,200	....
National, State, or County Buildings	1	519,400	....
Public schools	18	3,995,300	....
Street watering (estimated from data furnished by water register)	....	24,562,700	....
		31,275,500	3.898

NOTE. — For statistics of the consumption of water for public purposes in other cities and towns of the Metropolitan District, see Jour. New England Water Works Assoc., 1913, XXVII, pp. 125-131.

10. The Water Consumption for Manufacturing, Mechanical, and Trade Uses. — In Boston and certain other towns of the Metropolitan District this consumption was, in 1908, according to Dexter Brackett, as follows: *

* Jour. New England Water Works Assoc., 1913, XXVII, p. 107.

TABLE XI. — WATER SUPPLIED BY METER FOR MANUFACTURING, MECHANICAL AND TRADE USES, IN THE METROPOLITAN WATER DISTRICT

Boston, 1908 (population, 643,810)

Consumers		Number	Gallons	Per capita per day
Bakeries		24	18,517,100	0.079
Breweries and bottling		62	....	... ...
Massachusetts Breweries Co.	44,805,200			
Others	161,351,000			
		....	206,156,200	0.875
Cold storage plants		3	55,194,900	0.234
Contractors		2	2,685,300	0.011
Creameries		1	3,986,800	0.017
Dye works		7	5,565,400	0.024
Electric companies (Edison Elec. Ill. Co.)		1	190,687,600	0.809
Elevators and motors		340	138,178,100	0.586
Engines and boilers		8	3,111,700	0.013
Factories and machine shops		285	428,716,200	1.820
Farms and greenhouses		11	8,617,000	0.037
Fish packing houses		33	19,253,500	0.082
Gas works (Boston Consolidated Gas Co.)		1	114,900,300	0.488
Hotels		166	....	....
Touraine	46,076,800			
Parker House	37,549,600			
United States	36,210,700			
Adams House	32,687,600			
Others	568,195,800			
		....	720,720,500	3.059
Iron works		34	49,779,400	0.211
Laundries		71	154,505,900	0.656
Milk depots		2	22,679,400	0.096
Offices and stores		....	1,585,399,500	6.728
Oils and chemicals		8	9,731,500	0.041
Railways:				
Boston & Maine	459,997,600			
Boston Elevated	614,609,200			
Boston, Revere Beach & Lynn	45,740,200			
Boston Terminal Co.	125,866,000			
N. Y. Central & Hudson River	261,904,700			
N. Y., N. H. & H.	477,919,600			
Old Colony St. Ry.	381,500			
Union Freight R. R.	2,012,100			
			1,988,430,900	8.439
Restaurants		48	42,395,400	0.180
Rubber works		4	12,306,900	0.052
Saloons		104	46,733,600	0.198
Shipping, wharves, etc.		64	216,343,600	0.918
Slaughtering		2	24,968,200	0.106
Stables		339	169,242,500	0.718
Stone crushers		7	1,488,500	0.006
Stone works		6	19,605,100	0.083
Storage warehouses		27	26,157,600	0.111
Sugar refineries		1	246,196,700	1.045
Tanneries		5	3,366,000	0.014
			6,535,621,300	27.736

Somerville (population, 74,400)

Consumers	Number	Gallons	Per capita per day
Bakeries	7	913,300	0.033
Breweries	1	2,137,800	0.078
Coal dealers	1	668,700	0.025
Contractors	2	801,900	0.029
Dye works	2	4,730,400	0.174
Electric companies	1	374,700	0.014
Elevators and motors	8	1,765,300	0.065
Factories and machine shops	43	37,133,000	1.364
Farms and greenhouses	2	750,200	0.028
Hotels	10	4,731,800	0.174
Iron works	1	112,700	0.004
Laundries	5	12,431,000	0.456
Offices and stores	535	19,669,100	0.722
Oils and chemicals	1	712,100	0.026
Railways	2	....	....
Boston & Maine 148,789,900			
Boston Elevated 2,964,300			
		151,754,200	5.573
Slaughtering	3	....	....
North Packing & Provision Co. 92,847,700			
John P. Squire 59,820,600			
N. E. Dressed Meat & Wool Co. 43,627,800			
		196,296,100	7.209
Stables	57	17,805,200	0.654
		452,787,500	16.628

Malden (population, 42,140)

Consumers	Number	Gallons	Per capita per day
Bakeries	3	229,600	0.015
Coal dealers	11	1,302,300	0.084
Contractors	2	449,600	0.029
Dye houses	1	768,200	0.050
Electric companies	1	19,149,500	1.242
Elevators and motors	2	1,828,300	0.119
Factories and machine shops	34	16,491,200	1.069
Farms and greenhouses	15	1,371,900	0.089
Gas works	2	14,731,100	0.955
Hotels	3	806,300	0.052
Laundries	6	6,093,200	0.395
Offices and stores	278	25,882,600	1.678
Oils and chemicals	2	3,600,900	0.234
Railways	3	2,460,200	0.160
Rubber works	2	3,197,700	0.207
Shipping	1	44,100	0.003
Stables	22	2,330,800	0.151
Stone works	3	66,600	0.004
Tanneries	1	334,400	0.022
		101,138,500	6.558

Medford (population, 21,920)

Consumers	Number	Gallons	Per capita per day
Brick yards	2	2,375,500	0.296
Dye works	2	5,034,500	0.627
Factories and machine shops	8	3,192,800	0.398
Farms and greenhouses	7	845,000	0.105
Hotels	1	640,400	0.080
Iron works	1	955,800	0.119
Laundries	2	1,155,100	0.144
Offices and stores	131	3,847,900	0.480
Oils and chemicals	2	19,720,400	2.458
Railways	2	12,723,300	1.586
Stables	13	2,107,100	0.263
		52,597,800	6.556

NOTE. — For statistics of the consumption in other cities and towns of the Metropolitan District see Jour. New England Water Works Assoc., 1913, XXVII, pp. 112–116.

11. The Water Consumed in Public Schools and for Street Sprinkling was, in the Metropolitan District of Boston in 1901, as follows:

TABLE XII. — WATER USED FOR SCHOOLS AND STREETS IN METROPOLITAN DISTRICT *

City or town	Schools			Street sprinkling, 1901, Metropolitan district, gals. per capita per day
	Year	Daily consumption		
		Per pupil	Per inhabitant	
Arlington	1901	....	....	2.12
Belmont	1902	6.96	0.48	1.47
Boston	1899	6.22	0.41	2.35
Chelsea	....	....	....	1.12
Malden	1901	2.21	0.17	1.87
Medford	1901	4.02	0.33	3.41
Melrose	....	....	....	2.06
Milton	1901	6.56	0.53	5.87
Nahant	....	....	....	8.63
Quincy	....	....	....	1.43
Revere	....	....	....	0.96
Somerville	1899	6.29	0.46	1.97
Stoneham	....	....	....	0.60
Swampscott	....	....	....	3.67
Watertown	....	....	....	3.67
Winthrop	....	....	....	2.48

* Jour. New England Water Works Assoc., 1913, XXVII, p. 121.

12. *Water Consumption in Public Schools of Syracuse, N. Y.* — The quantity of water consumed daily per pupil during three months of 1903 was as follows:

TABLE XIII. — WATER CONSUMPTION IN PUBLIC SCHOOLS OF SYRACUSE, N. Y.*

School	Average daily attendance	Per capita consumption in gallons		
		March	April	May
1. Jefferson	314	17.8	21.1	24.1
2. Grant	304	22.2	25.2	24.4
3. Townsend	481	5.4	5.7	5.5
4. Garfield	434	5.1	6.6	5.2
5. Franklin	826	4.1	3.5	2.4
6. Prescott	574	9.4	9.2	6.0
7. Clinton	522	16.2	15.3	14.7
8. Lincoln	414	9.1	8.0	7.8
9. Vine	218	2.3	4.2	4.1
10. Frazer	428	12.8	14.2	11.0
11. Genesee	412	6.0	3.2	2.9
12. Commercial	86	397.0	381.0	497.0
13. May	426	110.0	114.0	117.0
14. Tompkins	388	5.6	6.1	8.7
15. Porter	778	4.5	4.5	3.2
16. Gere	352	33.0	33.5	22.5
17. Madison	529	40.7	39.4	32.8
18. Summer	358	20.9	20.0	15.7
19. Washington Irving	439	35.3	30.9	22.6
20. Willard	81	188.8	190.0	104.4
21. Montgomery	471	5.1	4.1	4.0
22. Putnam	650	21.6	16.7	12.6
23. Andrew Jackson	353	3.5	2.9	3.9
24. Croton	605	44.5	31.2	28.0
25. New High School	1267	8.9	9.4	9.0
26. Seymour	665	26.3	23.6	23.6
27. Truant school	16	22.3	38.2	52.2
28. Delaware	523	7.5	9.2	6.6
29. Grace	259	16.8	24.3	30.9
30. Merrick	294	35.6	17.9	22.3
31. Bellevue	257	32.1	32.8	37.9
32. Danforth	348	25.9	21.4	22.0
33. Elmwood	224	8.4	14.5	14.5
34. Brighton	868	5.3	4.1	3.0

* Average daily consumption was 20 gals. per capita. Jour. New England Water Works Assoc., 1913, XXVII, p. 122.

13. Consumption Based upon Area of Floor Space. — In small communities estimates of the probable consumption of water are properly based upon the resident population. For larger centers of population, the quantity of water required for industrial and commercial uses may affect considerably the average daily per capita consumption.

In important cities there is often a large transient population, consisting of people who come to the city for their daily work, but who reside in neighboring villages or towns. The water used by these people is usually charged in water works statistics, for lack of definite information, to the resident population, whose average daily per capita consumption is thus often largely increased. It is difficult to estimate the transient population in a city, but it is comparatively easy to figure the floor space in a city, occupied both by the resident and the transients in residences, hotels, offices, tenement houses, etc. For this reason, estimates of the probable consumption of water in a large city can be more correctly computed on a consumption per 1000 sq. ft. of floor space, than on a daily average per capita consumption, based upon the resident population.

The former plan was adopted by the Board of Water Supply of the City of New York, in studying how the water supply from the Catskill Mountains, obtained in 1905 to 1917, should be distributed in the different boroughs of Greater New York.* Measurements of the consumption in different parts of the city, which were made in 1902–3, before efforts had been made to reduce the waste of water, showed that the daily per capita consumption varied from 37 to 860 gals. Gaugings made in 1911 in the borough of Manhattan, the principal part of the city, gave the per capita consumptions contained in the following table:

TABLE XIV. — GAUGINGS OF 1911 IN BOROUGH OF MANHATTAN

No. of district	Consumption, million gallons daily	Population, resident	Consumption per capita
1. East side tenement, some water front	11.44	230,500	50
2. All classes	29.48	204,557	144
3. High class apartment and residence	22.18	186,990	118
4. High class apartment, residences and tenements	12.74	138,800	92
5. East side tenement	8.28	84,580	98
6. High class apartment, residences, tenements, and water front	14.82	173,000	86
7. All classes	13.38	169,100	79
8. All classes	13.66	209,393	65

The engineers of the Board of Water Supply of New York made estimates of the floor space, including basements, contained in different districts into which they divided the city. The daily water consumption in the different districts was found to vary generally only from 150 to 300 gals. per 1000 sq. ft. of floor space — a much closer agreement than when the consumption is based upon the resident population. This method may, doubtless, be used to advantage in other large cities.

14. Changes in Population. — In making plans for the water supply of a community, the engineer must not only consider the existing population, but, also, what it will probably be at some future period — usu-

* Proc. Amer. Water Works Assoc., 1912, p. 35.

ally taken twenty to thirty years after the installation of the works. The probable change in population can be estimated from census reports, when they are available, and from the experience of other communities of about the same size and similarly situated. The estimates of future population may be made by mathematics or graphically. If the former method is adopted, the change in population can be assumed to be a constant number for each year of the period under consideration, or a fixed percentage of the preceding year for each year of the period; in other words, the increase or decrease in population may be assumed to form an arithmetical or a geometrical progression. In the former case the population (P) at a number of years (n) after the installation of the works can be computed by the following formula, in which A is the population when the works are installed, and a is the probable annual increase or decrease in population, fixed by a study of the preceding years:

$$P = A \pm na. \tag{1}$$

If the change in population is assumed to follow a geometrical progression, the population at n years after the completion of the works will be:

$$P = A\ (1 + 0.01 \cdot p)\ n \tag{2}$$

where p is the assumed per cent of increase or decrease per annum.

A graphic representation is often used for estimating the probable changes in population. Whatever method is adopted, careful attention must be paid to changes due to special conditions, such as the annexation of suburbs by a city, the establishment of an important industry, etc.

In designing water works for a large community, the engineer has not only to study the increase or decrease in population of the city or town, as a whole, but he must, also, investigate what changes in population are likely to occur in the different parts of the community. For example, that part of Greater London known as "the City of London," which contains a little over a square mile in area, has been steadily losing resident population for more than a century, as will be seen by the following figures:

In 1801 the population of "the City of London" was 28,130
In 1902 the population of "the City of London" was 26,920
In 1911 the population of "the City of London" was 19,657

This decrease in resident population is due to the fact, that as facilities in transportation increased, the citizens moved to the suburbs, leaving the "old city" for business purposes. As far as water consumption is concerned, the demand became greater, owing to the large transient population in the business districts.

In some communities the natural growth in population is diminished by a reduction in the birth rate. Thus, while the population of the City of Munich, Germany, increased from 490,000 in 1900 to 590,000 in 1910, there was a steady reduction in the birth rate from 37 per 100 inhabitants in 1900 to 24.3 per 100 inhabitants in 1910.*

From what has been said it is evident that the growth in populations does not follow mathematical laws, and requires careful study.

* Otto Lueger's "Wasserversorgung der Städte, " Vol. 1 (2nd edition), p. 116.

CHAPTER II

FLOW OF WATER IN AQUEDUCTS

15. Aqueducts. — In its broad sense, the word aqueduct includes all kinds of channels in which water can be conveyed, such as ditches or canals, lined or unlined; flumes, built of wood, masonry or metal plates; masonry conduits, closed or uncovered; tunnels or pipe-lines. The same formulas for determining the flow of water may be used, with proper modifications, for all kinds of aqueducts. For convenience, we shall discuss the flow of water in pipes in a separate chapter, and shall devote the present one to the other kind of conduits.

16. The Chezy Formula. — In 1775 the French engineer Chezy proposed the following formula for the flow of water in open canals:

$$a\frac{h}{l} - \frac{pv^2}{c^2} = 0, \qquad (3)$$

where a = area of canal;
h = total fall;
l = length of canal;
p = wetted perimeter;
c = a constant, found by experiment.
v = velocity per second.

By transposing, and replacing $\frac{h}{l}$ by s, which represents the sine of the slope, and putting r, the mean hydraulic radius, for $\frac{a}{p}$, we obtain the Chezy formula as it is generally written, viz.:

$$v = c\sqrt{rs}. \qquad (4)$$

This formula has been used extensively for the flow of water in rivers, canals, pipes and aqueducts. The coefficient c varies, however, with r and s, and, also, according to the roughness of the wetted perimeter. The value of c has, therefore, to be found empirically for every particular class, size and slope of conduit.

17. The Kutter Formula. — Various improvements in the Chezy formula have been suggested. The formula proposed by the Swiss engineers Ganguillet and Kutter in 1869, based upon a careful study of all experimental data, which were then available, is used very much for flow of water in open canals, aqueducts and pipes. This formula, which is called in the United States, for brevity's sake, the Kutter formula, is as follows:

$$v = \left[\frac{41.65 + \frac{1.811}{n} + \frac{0.00281}{s}}{1 + \left(41.65 + \frac{0.00281}{s}\right) \frac{n}{\sqrt{rs}}} \right] \sqrt{rs}, \tag{5}$$

where n = coefficient of roughness, depending upon the nature of the wetted perimeter.

The following table gives values of n, found by experiments:

TABLE XV. — COEFFICIENTS OF ROUGHNESS n IN THE KUTTER FORMULA

Description	Lining	n
Open channel with uniform cross-sections....	Planed timber......	0.009
" " " " "	Neat cement........	0.010
" " " " "	Unplaned timber....	0.012
" " " " "	Brickwork..........	0.013
" " " " "	Rubble masonry....	0.017
" " " " "	Firm gravel........	0.020
" " " " "	Ordinary earth.....	0.025
" " " " "	Earth with stones, weeds, etc.........	0.030
" " " " "	Earth or gravel in bad condition.....	0.035
Large pipe, new and extra smooth...........	Cast iron...........	0.011
" " " " smooth.................	Cast iron or wood...	0.0125
" " riveted..........................	Sheet steel..........	0.0140

Of late years a great many experiments have been made on the flow of water through various kinds of channels and pipes. Many values of n for use in the Kutter formula, based upon these experiments, are given by Etcheverry.* The following tables have been prepared from these data:

* "Irrigation Practice and Engineering," by B. A. Etcheverry, Vol. II, pp. 48–55.

TABLE XVI. — COEFFICIENTS OF ROUGHNESS n IN THE KUTTER FORMULA

I. For Unlined Canals in Earth or Rock

Material of canal bed and sides	Condition	n
1. Indurated clay, brule clay, or soft shale	*a*	0.0150
2. Coated with a heavy layer of silt or sediment	*a*	0.0160
3. Stiff tenacious clay; volcanic ash soil, hardpan	*b*	0.0175
4. Clay loam; worn smooth adobe; firm sandy coarse soil; cemented gravel; compact small gravel	*c*	0.0200
5. (*a*) Sandy and clay loam soils; volcanic ash soils	*d*	0.0225
(*b*) Newly cleaned, plowed and harrowed	....	0.0225
6. (*a*) Mixed compact gravelly soil or gravel, varying up to about 3 ins. in diameter; loose sandy soils, forming sand dunes or ripples on the bottom	*e*	0.0250
(*b*) In sandy loam, or clay loam	*f*	0.0250
(*c*) Loose slate or shale rock, with projections 3 to 6 ins.	....	0.0250
7. (*a*) Loose, large size gravel up to 6 ins. in diameter; disintegrated rock or rough hardpan, with small scattered fragments	....	0.0275
(*b*) Earth below average fair conditions	*g*	0.0275
8. (*a*) Rough gullied hardpan, with eroded irregular cross-section, blast holes; seamy granite, or broken slate, with projections of 3 to 12 ins.	....	0.0300
(*b*) Earth in poor condition	*h*	0.0300
9. (*a*) Earth in poor condition	*i*	0.0350
(*b*) Large size loose gravel and cobbles	*i*	0.0350
10. (*a*) Rough scoured beds, cross-section about half filled with aquatic plants	*j*	0.0400
(*b*) Uneven surfaces of loose large size cobbles	....	0.0400
(*c*) Grass and brush slopes	....	0.0400
11. Bottoms entirely covered with dead moss, or entirely grass-lined	....	0.0500
12. In very poor condition	*k*	0.0750
13. In extremely bad condition	*l*	0.1000

a. In excellent condition, with well graded, smooth surfaces, or worn smooth by the water; uniform cross-section and regular alignment; free from sand, gravel, pebbles and vegetation.

b. In good condition, with well graded, smooth surfaces; uniform cross-section and regular alignment; free from gravel, cobbles, pebbles, and vegetation.

c. In good average condition; uniform cross-section and regular alignment; free from large gravel; with little loose gravel, pebbles or sand; practically free from seed and moss.

d. In average condition; small variations in cross-sections and fairly regular alignment; small amount of gravel, or a few small cobbles; few aquatic plants, thin growth of grass along edges and a little moss.

e. In average fair condition; waving banks with vegetation in the shallow water near the edges.

f. In a little below average fair condition; with overhanging sodded banks, roots or weeds along the edges, trailing in the water.

g. Variations in cross-section with eroded indentations on the sides, and either with scattered rocks or a few cobbles or loose gravel on the bottoms; or with considerable weeds or vegetation on the sloping banks.

h. Irregular cross-section with few aquatic plants and moss on the bed and overhanging trailing weeds, or vegetation on the sides and moss.

k. Thick vegetation on the banks trailing in the water, and dense growth of aquatic plants on the bottoms.

l. For canals with channels completely filled with cat tails, scattered willows, and trailing moss, with considerable dead water.

II. Concrete-lined Canals

Kind of lining	Condition	n
1. (*a*) Lining with smooth hard mortar finish like side-walk surface	*a*	0.012
(*b*) Wet mortar poured between forms of matched lumber, and painted or troweled smooth with cement mortar	*a*	0.012
2. (*a*) Lining built of separate sections or slabs cast in steel forms and joined in place	*a*	0.0125
(*b*) Lining built without forms, but *screeded* with rich grout to true surface	*a*	0.0125
3. Lining built with forms and partly troweled or floated with cement mortar	*a*	0.013
4. Lining built without forms, *screeded* to true surface and floated to give fairly smooth sand finish	*b*	0.014
5. Lining built with forms of planed lumber, partly roughened in construction, not plastered or grouted	*c*	0.015
6. Lining built with forms of rough lumber, no other finish	*c*	0.016
7. Lining built without forms, rough construction, with wavy surface made fairly smooth by tamping with the back of shovel	....	0.017
8. Lining as in (7), but with considerable sand or gravel in the bottom, or as in (5) and (6), with slime or moss deposit	....	0.018

a. Long tangents; no curves for velocities exceeding 4 ft. per second, and flat tapered curves for velocities less than 4 ft. per second.

b. Alignment conditions slightly inferior to *a*.

c. Alignment conditions slightly inferior to *b*.

III. Wooden Flumes

Kind of material	Condition	n
1. Lined with new surfaced lumber, in excellent condition	*a*	0.0115
2. Lined with new surfaced lumber, in excellent condition	*b*	0.0125
3. (*a*) Lined with surfaced lumber in average good condition; straight	*c*	0.0130
(*b*) Lined with rough lumber, worn smooth by sediment and fine sand in water; straight	*c*	0.0130
4. Lined with surfaced lumber in poor condition	*d*	0.0150
5. Lined with new rough lumber	*e*	0.0160
6. (*a*) Wooden lining in poor condition	....	0.0180
(*b*) Roughly asphalted, or projecting caulking	....	0.0180
(*c*) Thickly slimed	*e*	0.0180
7. Lined with wood, floor neatly covered with sediment, sand gravel or scattered rocks; or with considerable slimy moss	....	0.020

a. Longitudinal boards; smooth battens or smooth joints with no projecting caulking; straight alignment; no silt or deposit.

b. Slightly inferior to (*a*), or with flat curves or change in alignment by small angles, or straight with smooth coat of tar; no silt or deposit.

c. Straight, no silt or deposit.

d. Over 10 years old, or with numerous changes in alignment made by sharp angles with short tangents (200 ft. or less), or with old transverse flooring.

e. Not worn smooth, or with many short tangents.

In all cases where there is a deposit of sand, gravel, cobbles or rock, the value of n given in the above table is limited to velocities of less than 3 ft. per second.

*IV. Semi-circular Sheet Steel Flumes**

Kind of lining	Number of experiment	Velocity in feet per second	Average value of *n*
1. Countersunk joints, flush with inner surface.	8	1.66–6	0.0115
2. With projecting bands or ribs at each joint..	4	Under 2	0.013
3. With projecting bands or ribs at each joint..	4	2.00–3.50	0.016
4. With projecting bands or ribs at each joint..	5	4–6	0.018
5. Corrugated sheets........................	3	1.91	0.022

* No silt or sediment on the lining.

While the Kutter formula is quite complicated, its use can be much facilitated by diagrams and tables. It is, also, used in the following simplified form, in which b = coefficient of roughness, varying from 0.12 to 2.44. For ordinary brickwork $b = 0.35$.

$$v = \left[\frac{100\sqrt{r}}{b + \sqrt{r}}\right]\sqrt{rs}. \qquad (6)$$

18. The Bazin Formula. — In 1865 the French engineer Bazin proposed a formula for the flow of water in open channels. In 1897 he improved this formula to agree with the additional experimental data which were then available. The improved Bazin formula is as follows:

$$v = \left[\frac{87}{0.552 + \frac{m}{\sqrt{r}}}\right]\sqrt{rs}, \qquad (7)$$

where m = coefficient of roughness.

This formula is simpler than Kutter's, and is independent of the slope. The following table gives values of m to be used in the Bazin formula.

TABLE XVII. — COEFFICIENT OF ROUGHNESS m FOR THE BAZIN FORMULA

Nature of channel lining	m
Planed timber or smooth cement..............................	0.06
Unplaned timber, well-laid brick or concrete..................	0.16
Ashlar, good rubble masonry or poor brickwork................	0.46
Earth in good condition....................................	0.85
Earth in ordinary condition.................................	1.30
Earth in bad condition.....................................	1.75

19. Dr. Lampe's Formula. — For conduits lined with smooth concrete or brickwork, this formula gives good results. It is as follows:

$$v = 167.89\, r^{0.694} s^{0.555}. \qquad (8)$$

This formula agrees very closely with the gaugings made in the New Croton Aqueduct, which is a brick-lined conduit, having a horseshoe section, equivalent in flowing capacity to a circular section of 14 ft. diameter. For the greater part of its length, the conduit is built on a grade of 0.7 ft. per mile, and is *not under pressure.*

20. Maximum Permissible Velocities in Conduits. — Most waters that are conveyed in conduits carry at times more or less sand or silt in suspension, and wash gravel or stones along their beds. If the velocity of the current is too small, sediment is deposited in the bottom or on the sides of the conduit. If the velocity is too great, the matter suspended in the water or washed along will erode the bottom and sides of the conduit. The channels should, therefore, be so designed that the velocities of the currents will be kept within certain minimum and maximum velocities.

Sedimentation will not take place to any extent, where the mean velocity of the current is about 2 to 3 ft. per second, and such a velocity will, also, prevent the growth of aquatic plants in the water channel. The following table gives, according to Etcheverry,* the maximum mean velocities which are safe against erosion.

TABLE XVIII. — MAXIMUM MEAN VELOCITIES SAFE AGAINST EROSION

Material	Mean velocity in feet per second
Very light pure sand of quicksand character	0.75– 1.00
Very light loose sand	1.00– 1.50
Coarse sand or light sandy soil	1.50– 2.00
Average sandy soil	2.00– 2.50
Sandy loam	2.50– 2.75
Average loam, alluvial soil, volcanic ash soil	2.75– 3.00
Firm loam, clay loam	3.00– 3.75
Stiff clay soil, ordinary gravel soil	4.00– 5.00
Coarse gravel, cobbles, shingles	5.00– 6.00
Conglomerates, cemented gravel, soft slate, tough hardpan, soft sedimentary rock	6.00– 8.00
Hard rock	10.00–15.00
Concrete	15.00–20.00

21. The Flow of Water through Covered Masonry Conduits. — The formulas given for open channels can be used for covered masonry conduits, but when the conduit is *under pressure,* the value of s must be determined by dividing the total available head between any two given points by the length of the conduit between these points, measured along the axis of the conduit.

When the conduit is *not under pressure* — that is, when the water surface is always below the crown of the cover — the maximum discharge occurs when the conduit is not quite full, as proved on page 33.

* Etcheverry's "Irrigation Practice and Engineering," Vol. II, p. 57.

22. *Maximum Discharge of a Circular Gravity Conduit.* — In Fig. 3, assuming the conduit not to be under pressure, let:

A = water area;
p = wetted perimeter;
r = radius of the conduit;
θ = central angle subtended by water surface.

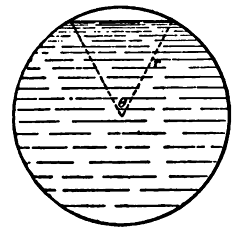


Fig. 3.—Maximum Discharge of a Gravity Aqueduct.

$$A = \frac{r^2}{2}[(2\pi - \theta) + \sin\theta].$$

$$p = r(2\pi - \theta).$$

$$\text{The hydraulic radius} = \frac{A}{p} = \left[\frac{(2\pi - \theta) + \sin\theta}{2\pi - \theta}\right]. \qquad (9)$$

Applying the differential calculus, we find that the hydraulic radius, and consequently the discharge of the conduit, will be a maximum when $\theta = 51° 48'$. The maximum theoretical flow in the circular conduit will, therefore, occur, when the depth of water $= 1.899\,r$, or, practically, when the water surface is $\frac{r}{10}$ from the crown of the intrados of the arch. Owing to disturbing conditions, such as eddies in the water, adhesion of the water to the wetted perimeter, slight irregularities in the masonry lining, etc., formula (9) will not always be found to be absolutely correct in practice.

CHAPTER III

FLOW OF WATER THROUGH PIPES

23. Flow of Water through Circular Orifices. — When water flows through a vertical or horizontal circular orifice in a thin wall, it contracts first, and then expands, as shown in Fig. 4. The jet, where it has its smallest diameter, is called the contracted vein (vena contracta). Numerous experiments have been made to determine the area of the contracted vein and its distance from the inner surface of the wall in which the orifice is made, and the discharge of the jet. The distance from the inner surface of the wall to the contracted vein is usually 0.50 to 0.80 of the diameter of the orifice, and the mean value of the discharge

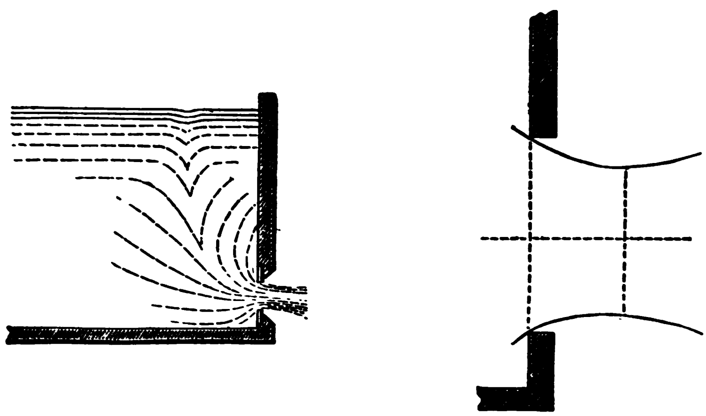


Fig. 4. — Flow of Water Through a Circular Orifice.

is about 0.62 of the theoretical discharge. The discharge from the orifice may, therefore, be expressed by the following formula:

$$Q = CA\sqrt{2gH}, \tag{10}$$

where Q = discharge, in cubic feet per second;
A = area of the orifice, in square feet;
C = coefficient of discharge = 0.62;
H = head in feet on center of orifice.

24. *The loss of head* caused by the flow of water through a circular orifice in a thin wall can be computed as follows:

Let H = head to center of orifice;
V = actual mean velocity at the orifice;
C = coefficient of discharge, assumed at 0.62.

Theoretically we should have $V = \sqrt{2\,gH}$.
We find by experiments $V = C\sqrt{2gH}$.

Hence $\frac{V^2}{2\,g} = C^2H$, and $H = \frac{V^2}{2\,gC^2}$.

If there were no loss of head, we would have $H = \frac{V^2}{2\,g}$.

We have, therefore: Loss of head $H - C^2H = (1 - C^2)\,H$, or expressed in terms of the velocity head

$$\text{Loss of head} = \frac{V^2}{2\,gC^2} - \frac{V^2}{2\,g} = \frac{V^2}{2\,g}\left(\frac{1}{C^2} - 1\right). \tag{11}$$

25. Flow of Water through Short Cylindrical Tubes. — If we substitute for a circular orifice a short cylindrical tube, having its inner surface flush with the inner surface of the wall, and a length of 2 to 3 times its diameter, the discharge will be increased. The jet will be at first contracted, as with an orifice, but as the jet expands and fills the tube the air around the contracted vein becomes entrained, and the area at the contracted vein expands so that the ratio of its area to that of the tube becomes about 0.815. The formula of discharge for a short cylindrical tube becomes, therefore, using an average value for the coefficient of discharge,

$$Q = 0.815\,A\sqrt{2\,gH}. \tag{12}$$

If the short tube is made bell-shaped (Fig. 5), so as to conform to the paths made by the particles of water in approaching the contracted vein, the coefficient of discharge for the smaller area of the tube becomes almost unity.

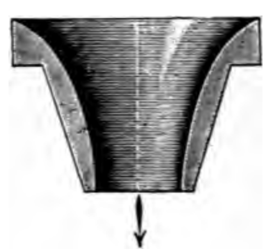


Fig. 5. — Bell-shaped Tube.

26. *Loss of Head.* — When water flows through a short cylindrical tube, the loss of head may be computed by formula (11), by making C = coefficient of discharge for short cylindrical tubes, assumed usually at 0.82. By using this value we get

$$\text{Loss of head} = \frac{V^2}{2\,g}\left(\frac{1}{0.82^2} - 1\right) = 0.487\,\frac{V^2}{2\,g}. \tag{13}$$

The head lost by the flow through a short cylindrical tube is, therefore, *practically half of the head causing the actual velocity V*.

27. Flow of Water through Long Pipes. — When the length of a pipe is more than 3 or 4 times its diameter, the resistance to flow, caused by the roughness of the inner surface of the pipe, has to be taken into consideration. Experiments have shown that this resistance, called friction, varies directly as the length of the pipe and the square of the velocity, and inversely as the diameter. It depends to a great degree on the character of the inside of the pipe. The smoother it is, the less the friction will be. The resistance in pipes, called friction, includes, besides the friction proper between the water and the sides of the pipes, which depends upon the viscosity of the water, the loss of energy resulting from eddies, caused by irregularities in the inner surface. If curves or gates occur in the pipe, they will cause additional loss of head. It has been found by experiments that for a length of 3 to 4 diameters from the inlet the flow in the pipe is similar to that in a short tube, that is, the water forms first a contracted vein and then dilates, so as to fill the whole pipe. A certain amount of head is lost at the entrance of a long pipe, as at that of a short tube. A part of the available head, equal to $\frac{v^2}{2g}$ is consumed in generating the velocity of the water flowing in the pipe. If we represent the total available head by H, and the various sub-heads required to overcome different resistances by:

h_e = entry head; h_v = velocity head;
h_f = friction head; h_c = head lost on curves;
and h_g = head lost at gates, etc., we shall have:

$$H = h_e + h_v + h_f + h_c + h_g. \tag{14}$$

The last two subdivisions of head are usually included in the friction head.

It is rational to suppose that the loss of head at the inlet of a long pipe will be similar to that which occurs at a short tube, although less in amount, on account of the slower velocity in a long pipe, compared with that of a short tube. It may, therefore, be expressed by equation (11), according to which

$$\text{Loss of head} = \left(\frac{1}{C^2} - 1\right)\frac{v^2}{2g},$$

where C = coefficient found by experiment, equal to 0.82 for flush ends, and 0.95 to 0.995 for curved ends.

Taking the usual case of the end of the pipe being flush with the side of the water basin, we get, including h_c and h_g in h_f:

$$H = 1.5\frac{v^2}{2g} + h_f. \tag{15}$$

In long pipe lines h_e and h_v are very small, so that practically the whole head may be considered as being consumed in overcoming the friction in the pipe. The formula for the flow in long pipes is, therefore, generally written

$$h = f\frac{lv^2}{d \times 2g}, \tag{16}$$

where h = total head; f = a friction factor, found by experiment; l = length of pipe; and d = diameter of pipe. From this equation we obtain, placing c for the constant $\sqrt{\frac{2g}{f}}$,

$$v = \sqrt{\frac{2ghd}{fl}} = c\sqrt{\frac{hd}{l}}. \tag{17}$$

28. Formulas for Flow in Long Pipes. — A great many empirical formulas for calculating the average velocity of water flowing through a long pipe have been devised. Henry Pitot, a French scientist, well known as the inventor of the Pitot tube, was the first to investigate the flow of water through pipes. He proved, about 1728, that the resistance to flow was inversely as the diameter. The first gaugings of the discharge of water pipes, of which we have any record, were made in France, prior to 1732, by Couplet at Versailles. Similar gaugings on the flow through pipes were made by Bossut at Mezières, France. Based upon the experiments of Couplet and Bossut, and some of his own, Dubuat was the first to propose a formula for the flow of water through pipes. Expressed in French inches,* which were used prior to the introduction of the metric system, it was as follows:

$$v^{\frac{7}{4}} = 133.6\,g\frac{d}{4}\left(\frac{h - \frac{v^2}{478}}{l}\right). \tag{18}$$

Chezy, Eytelwein, Prony, Weisbach, Hagen and other prominent hydraulicians endeavored to find better formulas for the flow of water in pipes, using all available data, but the laws of the flow of water in pipes were not clearly understood, until Henri Darcy made for the French Government in 1852–57 his famous experiments on flow of water through pipes. Darcy gauged the flow through new and old pipes of iron, lead, glass, and asphalt, having diameters of 0.0122 to 0.50 meters (0.48 to 19.68 ins.) and a length of 100 meters (328 ft.). The velocities of flow varied in these experiments from 0.16 to 5.0 meters (0.52 to 16. 4 ft.) per second. Darcy discovered from his gaugings the important fact that the resistance to flow in a water pipe depends largely upon the smoothness of the wetted perimeter. Based upon his experiments, Darcy obtained a formula, which has been extensively used, and is correct

* 1 French foot = 12 French inches = 0.3248 meter.

within the limits of his experiments, *i.e.*, for pipes less than 19.68 ins. in diameter. It is

$$v = \frac{1}{\sqrt{a + \frac{b}{d}}} \sqrt{rs}, \qquad (19)$$

where d is the diameter, and a and b are constants having the following values for English units:

For new cast iron pipes $a = 0.00007726$; $b = 0.00000647$.

For old or rough iron pipes $a = 0.0001545$; $b = 0.00001294$.

Darcy died in 1858, before his reports on his experiments were completed. His work was continued by his assistant Henri Bazin, who modified Darcy's formula for flow in pipes.

Other modern formulas for flow in pipes have been proposed by Lévy, Gauckler, Vallot, Flamant, Ganguillet and Kutter, Dr. Lampe, Manning, Prof. Unwin, etc.*

The complicated form of most of these formulas has led some engineers to use very extensively the simple Chezy formula (p. 27):

$$v = c\sqrt{rs}. \qquad (4)$$

As already stated, the main objection to this formula is the fact that the value of c varies with r, s and the character of the wetted perimeter.

In 1854 Hagen proposed the following equation for expressing the flow of water in pipes:

$$v = cr^a s^b, \qquad (20)$$

where r = mean hydraulic radius; s = sine of the hydraulic slope; and a, b, and c are numbers, found empirically, which remain constant for a given class of pipe or channel. A number of modern formulas for flow in pipes are based upon Hagen's equation. The principal of these formulas are given in the following table, and, also, a formula devised by the author in collaboration with Albert Ahrens, Jr. M. Am. Soc. C. E.

TABLE XIX. — EXPONENTIAL FORMULAS FOR FLOW IN CLEAN CAST IRON PIPES

Authority	Formula
St. Venant	$v = cr^{0.583}s^{0.583}$
Lampe	$v = 125.67\, r^{0.694}s^{0.555}$
Flamant	$v = 141.72\, r^{0.714}s^{0.571}$
Williams and Hazen	$v = 171.5\, r^{0.63}s^{0.54}$
Williams	$v = 171.0\, r^{0.668}s^{0.535}$
Barnes	$v = 174.1\, r^{0.769}s^{0.529}$
Wegmann and Ahrens	$v = 182.5\, r^{0.723}s^{0.539}$

* G. Dariès, "Calcul des Conduites d'Eau," pp. 15–23, Paris.

The last of the formulas in Table XIX was obtained by the author and Mr. Ahrens after a careful study of 260 experiments on the flow of water through clean cast iron pipes. In using these experiments, they were *weighted,* according to the care and accuracy with which they were made, the completeness of the data given, and the experience of the experimenter. The author hopes to explain soon the details of how the formula was derived, in a paper to be presented to the American Society of Civil Engineers. The formula proposed by Wegmann and Ahrens will be found to agree more closely with the weighted experiments than any of the others given in Table XIX.

29. Flow through Wooden Pipes. — E. A. Moritz, Assoc. M. Am. Soc. C. E., made careful tests of the flow of water through wood-stave pipes. For the Kutter formula he found the following values for the coefficient of roughness n.*

TABLE XX. — VALUES OF n FOR USE IN KUTTER'S FORMULA FOR WOOD STAVE PIPE

Diameter, inches	Year of experiment	Velocity in feet per second				
		0.5	1.0	2.0	3.0	4.0
4	....	0.0117	0.0111	0.0104	0.0099	0.0096
5	....	0.0105	0.0107	0.0106	....	....
6	1909	0.0103	0.0102	0.0100	....	....
6	1910	0.0103	0.0105	0.0102	0.0100	....
Mean 6						
8	1909	....	0.0115	0.0111	0.0109	0.0107
8	1910	0.0110	0.0103	0.0095	0.0091	0.0089
Mean 8						
12	....	0.0130	0.0128	0.0122	....	....
14	1909	0.0118	0.0114	0.0109	0.0105	....
14	1910	0.0104	0.0108	0.0108	0.0106	....
Mean 14						
18	....	0.0112	0.0109	0.0105	0.0103	....
22	....	0.0096	0.0107	0.0120	0.0126	....
$55\frac{3}{4}$	1909	....	....	0.0109	0.0105	....
$55\frac{3}{4}$	1910	....	0.0118	0.0113	0.0110	0.0108
Mean $55\frac{3}{4}$						

Based upon the experiments recorded in the above table, Mr. Moritz proposed the following exponential formulas for the flow through wood-stave pipes:

$$V = 1.72\, D^{0.7}\, H^{0.555}, \tag{21}$$

where V = velocity in feet per second;
D = diameter of pipe in inches;
H = friction loss in feet per 1000 ft.

* Trans. Am. Soc. C. E., 1911, LXXIV, p. 411; *Eng. Record,* December 13, 1913, p. 667.

According to Mr. Moritz, the above formula is correct within 10 per cent for velocities ranging from 0.5 to 6.0 ft. per second.

30. Loss of Head on Curves and Bends. — The first experiments on the additional loss of head caused in a water pipe by curves were made, about 1777, by Bossut* with a pipe 0.09 ft. diameter and about 53 ft. long. Bossut tried to find the diminished discharge of a pipe, caused by laying it in a serpentine line, instead of straight. Dubuat (1786), Venturi (1797), and Rennie (1831) also made experiments on the resistance caused by curves in pipes.

Weisbach undertook a series of experiments to ascertain the losses of head due to elbows and curves in small pipes, less than 2 ins. in diameter. He found that elbows caused much greater resistance to flow than curves, and that the resistance due to change of direction decreases, as the diameter of the pipe increases. Expressing the loss caused by curves in pipes by

$$h = n\frac{v^2}{2g}, \tag{22}$$

the values of n for different ratios of radius of the pipe to the radius of curvature were found by Weisbach to be as follows:

COEFFICIENTS OF RESISTANCE CAUSED BY CURVES IN PIPES

$\frac{r}{R} = 0.1$	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
$n = 0.13$	0.14	0.16	0.21	0.29	0.44	0.66	0.98	1.41	1.98

NOTE. — r = radius of pipe; R = radius of curve.

In 1888 John R. Freeman, M. Am. Soc. C. E., investigated the effect of curves in lines of fire hose† 2.49 and 2.64 ins. in actual diameter. The curves were complete circles, 2, 3, and 4 ft. in diameter. Expressing the resistance caused by the curve by the formula

$$h = f\frac{l}{d}\frac{v^2}{2g}, \tag{23}$$

where h = head lost on account of the curve; f = coefficient of friction; l = length of pipe; d = diameter of pipe; v = velocity in the pipe. The results obtained by Freeman were as follows, R representing the radius of the curve:

* "The Foundation of Our Knowledge of Hydraulic Curve Resistance," by Gardner S. Williams, M. Am. Soc. C. E., in *The Technic*, 1899, p. 48.

† Trans. Am. Soc. C. E., 1889, XXI, p. 363.

For Hose, 2.49 Ins. in Diameter

$\frac{R}{d} = 19.2$	14.4	9.6
f 0.0033	0.0034	0.0048

For the Hose, 2.64 Ins. in Diameter

$\frac{R}{d} = 16.2$	13.6	8.1
$f =$ 0.0036	0.0046	0.045

31. G. S. Williams, C. W. Hubbell, and G. H. Fenkell made careful observations* in 1897–1901 on the loss of head due to 90-degree curves on a line of new, tar-coated 30-in. cast iron pipe, which forms part of the distribution system of Detroit, Mich. Similar, though less extensive observations, were made later by these engineers on curves in lines of 16- and 12-in. cast iron pipes. The mean velocity of the water in the different pipes during the experiments was as follows:

For 30-in. pipe 0 to 3.5 ft. per second; 16-in. pipe 0 to 5.8; 12-in. pipe 0 to 4.8.

From these experiments, which were made with great care, Williams, Hubbell, and Fenkell came to the conclusion that curves of short radius, down to a limit of about 2½ diameters, offer less resistance to the flow of water than do those of longer radius, and hence that the theories and practices regarding curve resistance, as set forth in the hydraulic treatises of all nations up to the time of these experiments, were incorrect. They also found that the effects of disturbances of the flow of water in pipes are transferred for many diameters beyond the point where the interference occurs.

Denoting by R the radius of the curve, and by D the inner diameter of the pipe, the relative increased loss of head due to curves, in a length of 80 diameters, including the curves, was found to be as follows:

30-In. Pipe

$\frac{R}{D}$	2	2½	3	4	5	10	15	20	25
Per cent increase in loss of head......	14	13	14	18	24	50	67	80	93

* Trans. Am. Soc. C. E., 1902, XLVII, pp. 1–196.

12-*In. Pipe*

$\frac{R}{D}$	1.08	2	3	4
Per cent increase in loss of head	24.6	10.8	17.1	19.1

The conclusion of Williams, Hubbell, and Fenkell "that curves of short radius, down to a limit of about $2\frac{1}{2}$ diameters, offer less resistance to the flow of water than do those of longer radius" is not in accordance with the careful experiments made subsequently by A. V. Saph and E. W. Schoder, George J. Davis, Jr., and by Arthur W. Brightmore.

32. Saph and Schoder's experiments on the resistance on curves in pipes were begun in 1901 at the Cornell University Hydraulic Laboratory, while the experimenters were still students. Their attention was directed toward this line of experimentation by Prof. Gardner S. Williams, in charge of the laboratory, who was one of the engineers who made the above-mentioned Detroit experiments.

The first object of Saph and Schoder was "to find out, by means of Pitot tube traverses whether the normal conditions of flow of water in pipes were altered by passage around a curve, and in what manner the altered conditions resumed again the normal in the succeeding tangent." Along with this, they planned to perform comprehensive experiments in the loss of head due to 180-degree curves of various radii. The first tube experimented on was a 2-in. drawn brass pipe.* They found from 148 Pitot tube traverses that the disturbances caused by a smooth curve is not of the highly complex and indefinite nature, ordinarily ascribed to it, and that the point of maximum velocity is merely shifted towards the converse side of the pipe, this shifting causing, of necessity, a variation in the shape of the *velocity contours.* In normal flow in a straight pipe, these contours are concentric circles. The change in their shape caused by a curve is shown in Fig. 6, in which a symmetry about a horizontal axis exists, as might have been expected. The experiments showed that most of the distortion occured in the first quadrant, and comparatively little in the second (Fig. 7).

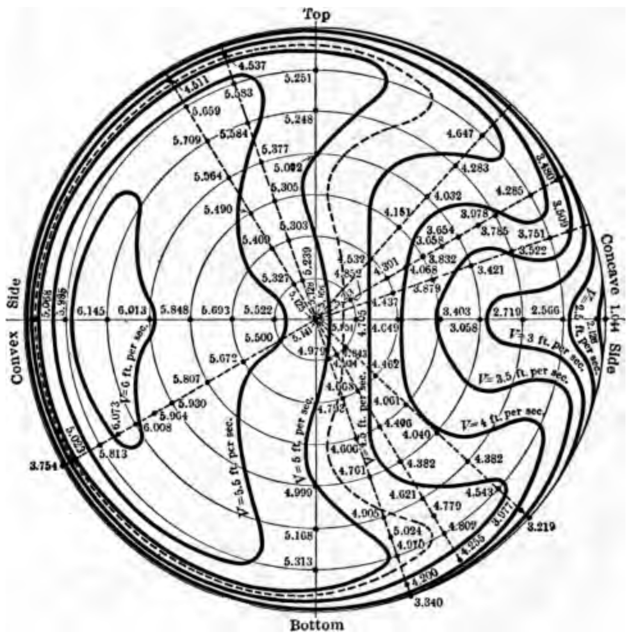
Thinking that large piezometric errors were involved in the Detroit experiments, Saph and Schoder undertook in 1907 a series of experiments on curve resistance in a 6-in. wrought iron pipe at the Cornell Hydraulic Laboratory.† These experiments contradict the general conclusions reached by Williams, Hubbell, and Fenkell from the Detroit experiments. Saph and Schoder found that for velocities of 3–5 ft. per

* Trans. Am. Soc. C. E., 1902, XLVII, p. 295.

† Trans. Am. Soc. C. E., 1909, LXII, pp. 67–96.

second the loss of head due to the curve was about the same for all radii, but when the velocity was increased up to 16 feet per second, the loss of head was increased by diminishing the radius.

33. Arthur W. Brightmore, M. Inst. C. E., experiments on curve resistance* were made with clean cast iron pipes, 3, 4, and 6 ins. in diameter. In order to reproduce, as nearly as possible, the condition



Diagram, showing velocity contours just beyond a 180° curve, in 2-in. brass pipe. The points plotted above are obtained from the formula $V=\sqrt{2gh}$, h being the observed difference of head (point and ring by Tube G) at the various points of the cross-section, at a distance 0.200′ (=1.66 diameters) from the P.T. of Curve No. 1 (Radius=1.659′=9.58 diameters.)
The extreme range of mean velocity (by weight) was 4.817 to 4.859 ft. per second. Average mean velocity=4.840 ft. per second (indicated in sketch by broken line.)

FIG. 6. — Velocity Contours Just Beyond 180° Curve, in 2-inch Brass Pipe. (Average Mean Velocity, 4.840 ft. per second, shown by broken line.)

of a cast iron pipe, which has been cleaned and from which the coating has been removed, Brightmore took uncoated pipes, and allowed their inner surfaces to become rusted, but not tuberculated. He first determined the coefficients in the Chezy formula for these pipes for straight lengths of 50 ft., which were as follows:

For 3-in. pipe, $c = 41.3$; for 4-in. pipe, $c = 47.5$; for 6-in. pipe, $c = 50.6$.

* Proc. Inst. C. E., 1907, CLXIX, p. 315.

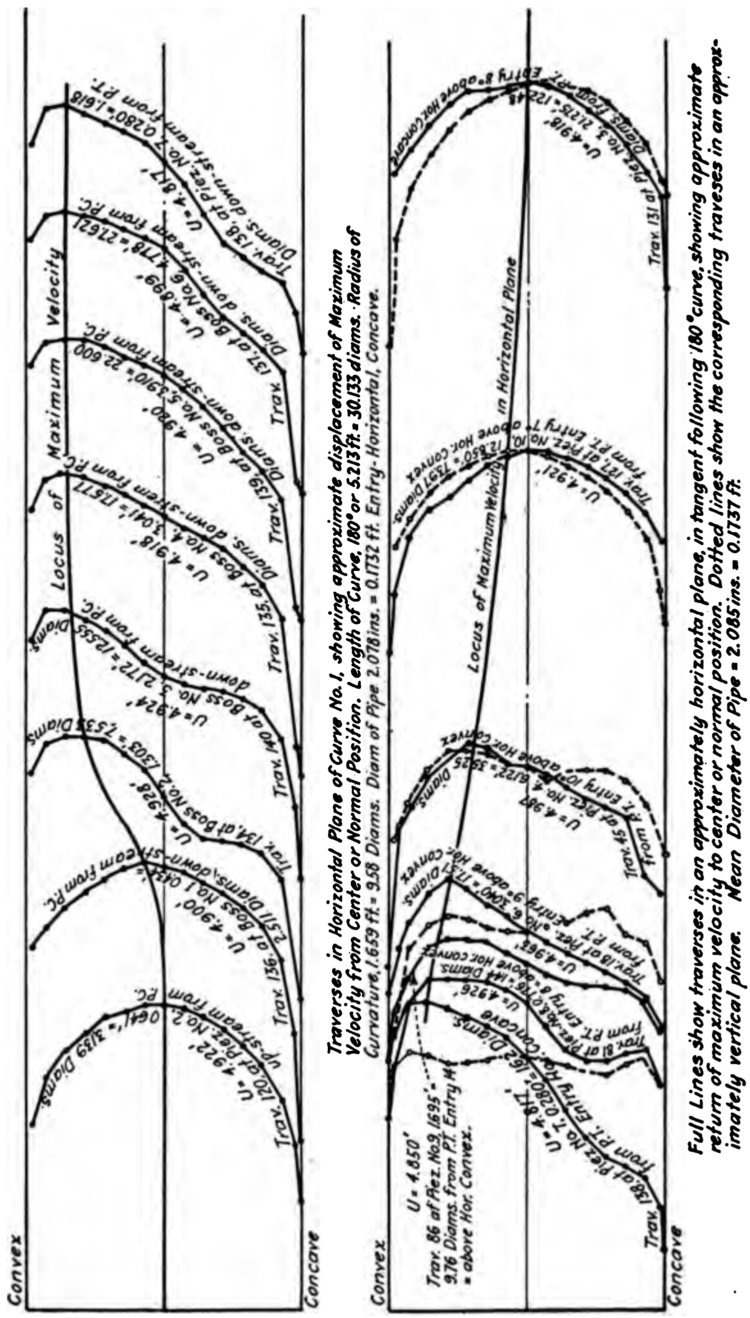


Fig. 7. — Distortion of Velocity Contours in 2-inch Brass Pipe on 180° Curve.

The results of the experiments on curve resistance, which are given in Fig. 8, are very similar to those obtained by Saph and Schoder.

34. George J. Davis, Jr., Assoc. M. Am. Soc. C. E., experiments on curve resistance were made in 1908 at the Hydraulic Laboratory of the University of Wisconsin.* The experiments were made with lap-welded wrought iron or steel pipes, $2\frac{1}{16}$ ins. in diameter. All of the curves made a turn of 90 degrees, and had radii, ranging from 0.788 to 10.00 times the diameter of the pipe. The results obtained were similar to those shown in the experiments of Saph and Schoder and of Brightmore.

35. Morrell's Conclusions about Curve Resistance. — An interesting study of the results obtained in the various experiments on the loss of

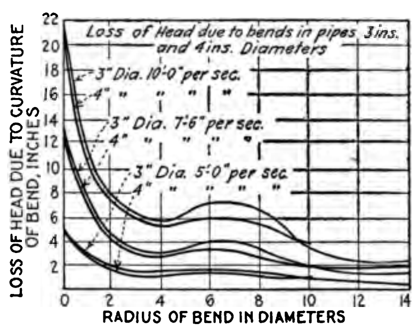


FIG. 8.

head caused by curves was made by Ben Morrell, Assistant Engineer on the Water Works of St. Louis, Mo.† From the data of the experiments of Weisbach, Saph and Schoder, Davis and Brightmore, he plotted for each velocity the losses of head found in the different experiments. From the diagrams thus obtained (Figs. 9 to 15), he assembled the average lines in Fig. 14, together with interpolated values up to velocities of 16 ft. per second. All the data plotted in Figs. 9 to 12 inclusive are for bends of 90 degrees.

Mr. Morrell did not include the experiments made by Williams, Hubbell, and Fenkel in his diagrams for the following reasons:

1. The extreme smallness of the measured losses, combined with the comparatively low velocities available (the greatest being about 3.5 ft.

* Trans. Am. Soc. C. E., 1909, LXII, pp. 97–112.
† *Eng. News*, Feb. 17, 1916, p. 302.

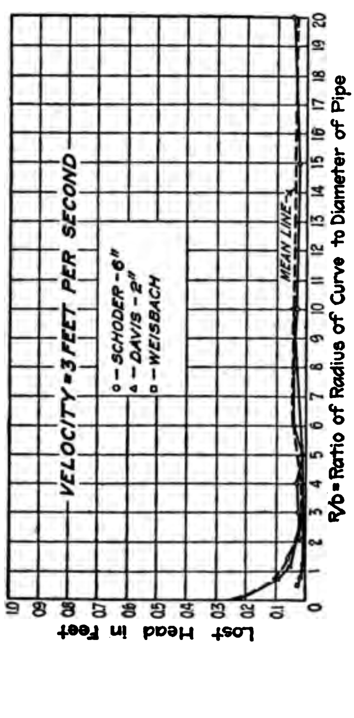


FIG. 9.

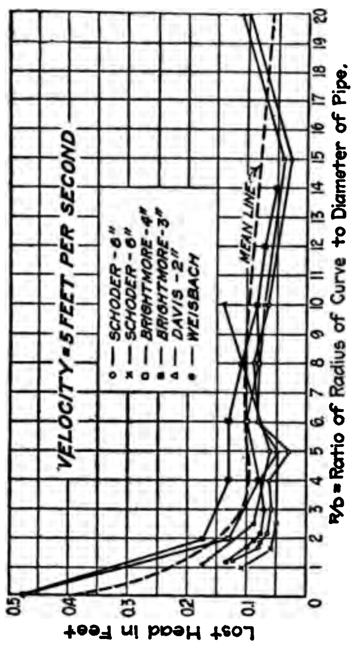


FIG. 10.

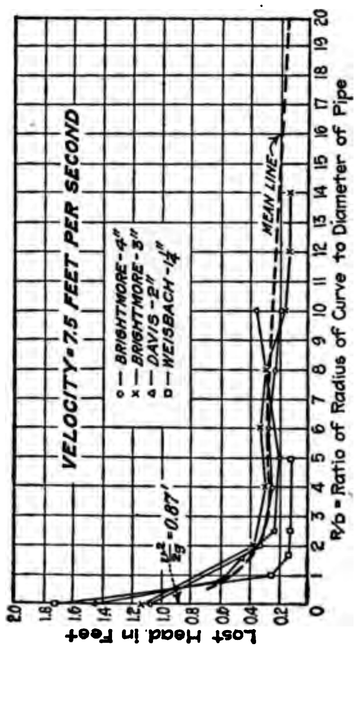


FIG. 11.

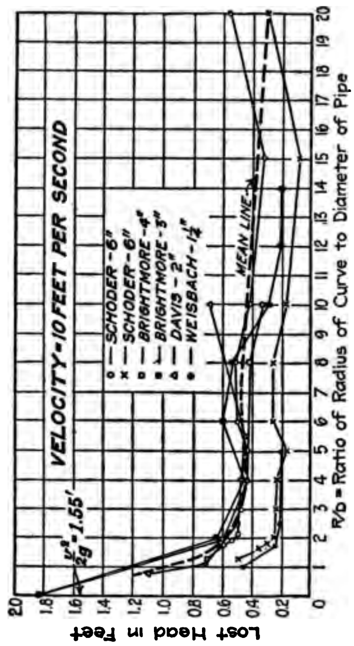


FIG. 12.

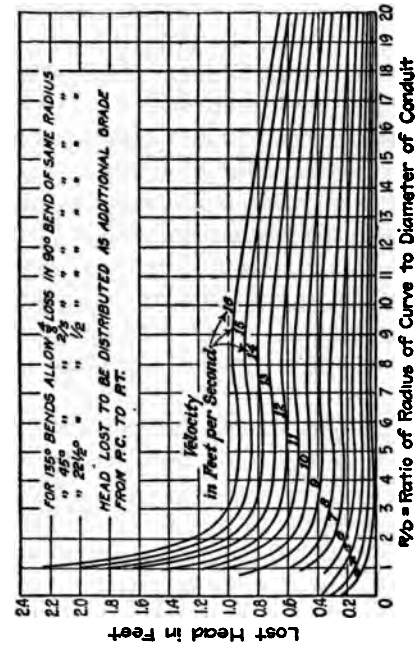

HEAD LOST TO BE DISTRIBUTED AS ADDITIONAL GRADE
FROM P.C. TO P.T.
Velocity
in Feet per Second
Lost Head in Feet
R/D = Ratio of Radius of Curve to Diameter of Conduit

FIG. 14.

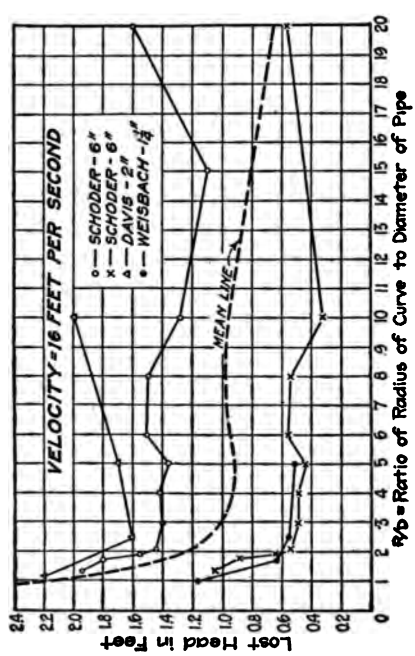

VELOCITY = 16 FEET PER SECOND
o — SCHODER - 6"
x — SCHODER - 6"
Δ — DAVIS - 2"
• — WEISBACH - 1¼"
MEAN LINE
Lost Head in Feet
R/D = Ratio of Radius of Curve to Diameter of Pipe

FIG. 13.

per second in the 30-in. pipe), would tend to magnify losses due to other effects than curvature.

2. The up-stream piezometers were placed where they would be affected by curves in the previous section.

3. The down-stream piezometers were placed too near to the point of tangency of a curve, to measure the total loss.

For a curve of 180 degrees, Saph and Schoder found that very little distortion takes place in the second quadrant of the curve. According to Weisbach's experiments, adding 90 degrees to a 90-degree curve of the same radius only increased the resistance about 40 per cent. The increased resistance was probably due to the greater length of pipe over which the maximum velocity was shifted toward the wall, increasing thus the frictional loss.

Weisbach also found that when a tangent was introduced between two curves turning in the same direction, the loss of head due to the second curve was increased because, as soon as the water reaches the tangent, the distortion begins to become less. The water has, therefore, not only to be redistorted in the second curve, but energy is also absorbed in the readjustment in the tangent. If the tangent between the curves is so short, that the distortion caused by the first curve is not entirely eliminated, the full loss, due to the two curves, will probably not occur. This question has, however, not yet been settled by experiments.

When a reversal of curvature takes place, the loss which would normally occur in the down-stream tangent of the first curve is evidently eliminated. On the other hand, a loss of energy will take place on account of the sudden shifting of the position of maximum velocity, with the accompanying eddy action, from one side of the pipe to the other. Until experiments show what the true loss is for this case, it would seem best to assume that the loss on a reverse curve is equal to that of the two separate curves. Saph and Schoder found in their experiments on a 6-in. pipe that the condition of curve distorted flow disappeared in a distance of 76 diameters down-stream from the point of tangency. Morrell recommends, however, in the case of a conduit operating at flow-line grade, to distribute the lost head due to curvature, as additional grades from the point of curve to the point of tangency. This will cause a portion of the straight conduit below the curve to be under a slight head, if it is assumed to be flowing full.

Based upon his studies, Morrell suggests the following rules for computations:

1. For bends of 90 degrees the lost head is to be obtained from the curves shown in Fig. 14.

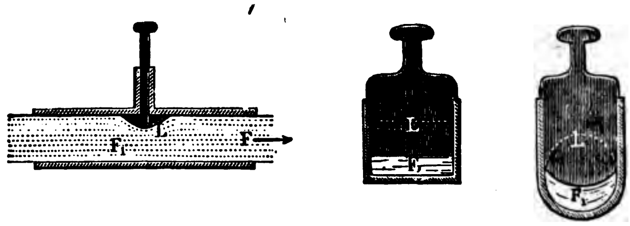
2. For various other bends allow as follows:

For 22½-degree bend allow ½ of loss of 90-degree curve of same radius.
For 45-degree bend allow ⅔ of loss of 90-degree curve of same radius.
For 135-degree bend allow ⅘ of loss of 90-degree curve of same radius.

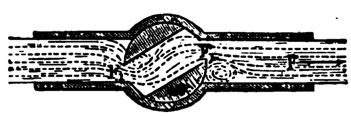
3. The tangents connecting the curves, turning in the same or in opposite directions, should be as short as possible.

4. The curve compensation in the case of a conduit operating at flow-line grade should be distributed as additional grade from the point of curve to the point of tangency.

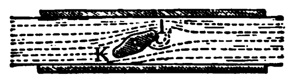
While the conclusions of Morrell are based to a large extent upon experiments made on comparatively small pipes, they seem justified by the existing data, and may be accepted until experiments on larger pipes



A — Sliding Gate-valves.



B — Stop-cock.



C — Throttle-valve.

Fig. 15.

shed more light on the intricate subject of loss of head occasioned by curves in pipes.

36. Loss of Head in Gate-valves. — In 1842 Weisbach published the results he had obtained in experiments on the losses of head caused by small gate-valves, stop-cocks and throttle-valves. These experiments appear to have been the first of their kind. Weisbach expressed this loss by the formula $h = n \frac{v^2}{2g}$, in which h = head lost; v = velocity per second in the pipe; g = 32.2; and n is a coefficient determined by experiment.

Fig. 15 shows the three kinds of valves with which Weisbach made his experiments: *A* shows sliding gate-valves; *B* is a stop-cock; and *C* is a throttle-valve or circular disc revolving on an axis. The values of n for these three types of stop-gates, placed in small pipes, less than 2 ins. in diameter, were found by Weisbach to be as follows:

TABLE XXI. — WEISBACH'S COEFFICIENTS OF RESISTANCE FOR GATE-VALVES, ETC., FOR CYLINDRICAL PIPES

Small Gate-valves

Ratio of height of opening to diameter	0	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$
Ratio of cross-sections	1.00	0.95	0.86	0.74	0.61	0.47	0.32	0.16
Coefficient of resistance n	0.00	0.07	0.26	0.81	2.06	5.52	17.0	98.0

Small Stop-cocks

Angle that the cock is turned	5°	10°	15°	20°	25°	30°	35°
Ratio of cross-sections	0.93	0.85	0.77	0.69	0.61	0.54	0.46
Coefficient of resistance n	0.05	0.29	0.75	1.56	3.10	5.47	9.68
Angle that the cock is turned	40°	45°	50°	55°	60°	65°	$82\frac{1}{8}$°
Ratio of cross-sections	0.39	0.32	0.25	0.19	0.14	0.09	0
Coefficient of resistance n	17.3	31.2	52.6	106	206	486	.....

Small Throttle-valves

Angle that the valve is turned	5°	10°	15°	20°	25°	30°	35°	
Ratio of cross-sections	0.91	0.83	0.74	0.66	0.58	0.50	0.43	
Coefficient of resistance n	0.24	0.52	0.90	1.54	2.51	3.91	6.22	
Angle that the valve is turned	40°	45°	50°	55°	60°	65°	70°	90°
Ratio of cross-sections	0.36	0.29	0.23	0.18	0.13	0.09	0.06	0
Coefficient of resistance n	10.8	18.7	32.6	58.8	118	256	751	∞

The values given by the above tables are only correct, when the water, after passing the contracted orifice, fills the pipe again. This requires a certain length of pipe at the outlet side of the gate-valve, etc.

No other results of similar experiments were published, until Emil Kuichling's paper on "Loss of Head, from the Passage of Water through a 24-inch Stop-valve" appeared in 1892.* Kuichling's experiments were made with great care, and, as might be expected, give different results than those obtained by Weisbach with small gate-valves.

In 1892 J. Waldo Smith, M. Am. Soc. C. E., made experiments on the loss of head, caused by a 30-in. gate-valve.† The following table gives the values of the coefficient c in the formula $Q = cA\sqrt{2gh}$ as deduced by Kuichling from these two sets of experiments. In this formula Q = volume discharged; A = area of opening; h = head lost in the valve.

TABLE XXII. — COEFFICIENTS FOR LARGE GATE-VALVES

Ratio of height of opening to diameter	0.05	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80
Ratio of area of opening to total area	0.05	0.10	0.23	0.36	0.48	0.60	0.71	0.81	0.89
Coefficient c for 24-in. valve	1.7	1.0	0.72	0.70	0.77	0.92	1.2	1.6	...
Coefficient c for 30-in. valve	1.2	0.9	0.83	0.82	0.84	0.90	1.05	1.35	2.1

* Trans. Am. Soc. C. E., 1892, XXVI, pp. 439–452.

† Ibid., 1895, XXXIV, pp. 235–243.

In 1899 some experiments on the flow through various kinds of small valves were made by Prof. W. T. Magruder of the Ohio State University.* Where wide open, the coefficients of discharge for gate-valves were found to be 0.5 to 0.7, and for globe-valves they were 0.3 to 0.4.

37. Head Lost by Abrupt Enlargement or Contraction of Section. — If an abrupt change is made in the cross-section of a conduit conveying water, either by enlarging or contracting it (Figs. 16 and 17), a certain loss of head will occur. Theoretical formulas for these losses are to be found in all books on hydraulics, but there are not enough experimental data at hand for a practical formula. If the change of area is made gradually — as by a reducer, in the case of a pipe — the loss of head, where such changes occur, will be trifling. This course should, therefore, always be followed in practice, if possible.

An interesting series of tests on the loss of head, due to sudden enlargement in circular pipes, was made by W. H. Archer, in 1912, at the Hydraulic Laboratory of the University of California.‡ Mr. Archer

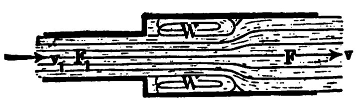


FIG. 16. — Enlargement of Pipe Section.

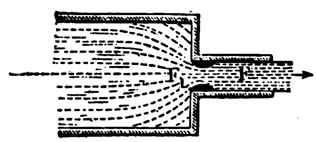


FIG. 17. — Contraction of Pipe Section.

gives the theoretical formula for the loss of head, caused by the enlargement as

$$h = \left(\frac{v_1 - v_2}{2\,g}\right)^2, \tag{24}$$

or in another form,

$$h = \frac{v_1^2}{2\,g}\left(1 - \frac{a_1}{a_2}\right)^2, \tag{25}$$

where h = theoretical lost head, in feet of water, due to sudden enlargement;

v_1 = velocity in feet per second, at outlet of smaller pipe;

v_2 = velocity in feet per second, in large pipe, at section where the pressure per square inch is a maximum;

g = 32.2;

a_1 = cross-sectional area, in square feet, at outlet of smaller pipe;

a_2 = cross-sectional area, in square feet, where the maximum pressure per square inch occurs.

* *Eng. Record*, June 24, 1899, p. 78.

† Trans. Am. Soc. C. E., 1913, LXII, p. 999.

The pipes experimented on were smooth brass pipes, 4 ft. long. The ends were machined square and even, and the pipes were connected with cast iron flanges, bolted together with gaskets between. The results obtained were as follows:

For the head actually lost on account of the enlargement, the formula becomes

$$h = 1.098 \frac{v_1^{1.919}}{2g}\left(1 - \frac{a_1}{a_2}\right)^{1.919}, \tag{26}$$

or

$$h = 1.098 \frac{(v_1 - v_2)^{1.919}}{2g}, \tag{27}$$

or

$$h = \frac{B\,(v_1 - v_2)^2}{2g}, \tag{28}$$

where B is a variable coefficient, ranging in the experiment from 0.754 to 1.225.

The distance of maximum pressure from the plane of enlargement, denoted by X, was found to be given by the following formula:

$$X = 1.45\,(d_2 - d_1)^{0.4}, \tag{29}$$

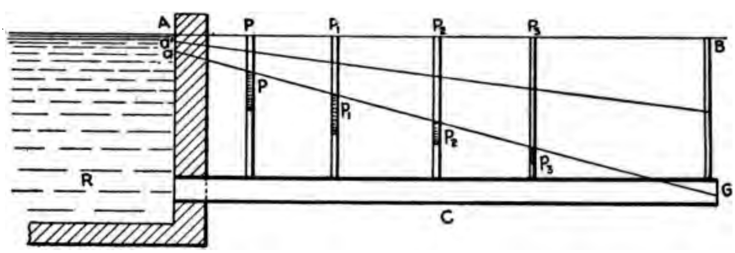


FIG. 18.

where d_2 and d_1 are respectively the diameters of the large and of the small pipe in inches.

38. Hydraulic Gradient. — Fig. 18 shows a straight horizontal pipe C, drawing water from a reservoir R. Small vertical pipes P, P_1, P_2, etc., called piezometers, are attached to the main pipe at regular intervals, the first piezometer P being placed at the reservoir. The pipe has a gate G at its end.

If the gate is closed, the water in all of the piezometers will stand in the horizontal line AB at the level of the water in the reservoir. If the gate is opened wide, the water in the different piezometers will drop to the inclined line aG which is called the hydraulic gradient. The distance Aa from the surface in the reservoir to the beginning of the hydraulic gradient is the head lost in elocity of the water in the pipe and in overcoming ance. The difference between Pp and P_1p_1 represents the loss of head caused by the friction of the water in the pipe between piezometers P and P_1. As the latter loss is directly proportional to the length of the pipe, the line aG, to which the water in the different piezometers rises, must be straight. If the gate is now partially closed, the water will stand higher in the different piezometers, and the hydraulic gradient will assume some intermediate position $a'G'$ between the line aG for open gate and the horizontal line AB, approaching the latter more and more, as the gate is gradually closed.

Should a curve, gate, reduction or enlargement of diameter occur in the pipe-line between any two consecutive piezometers, there will be an additional loss of head, which will cause the hydraulic gradient to be on a steeper slope between these piezometers than between the others. The hydraulic gradient thus becomes a broken line.

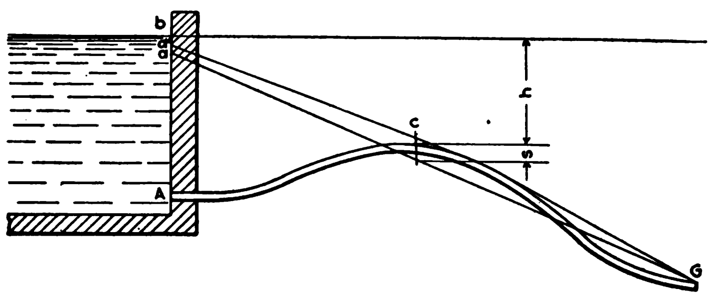


FIG. 19.

In actual practice, a pipe line will have both horizontal and vertical curves, gates, changes of diameter, etc., and the hydraulic gradient will, therefore, be an imaginary broken line, vertically above the pipe, to which the water in the pipe would rise at any given point, if a piezometer were attached there.

Occasionally a pipe will be laid so as to rise at one or more points above the hydraulic gradient (Fig. 19). In such cases the water is carried over the rise at C by siphonic action, providing the height s is less than that of a column of water, equal to the atmospheric pressure. Theoretically, s should not exceed 34 ft., but practically it should be less — say about 20–24 ft. — depending upon the velocity of the water, the tightness of the joints, etc. At C a minus pressure, *i.e.*, suction, would occur.

If s is greater than 34 — or, in actual practice, more than about 24 ft. — the water will not be siphoned above the hydraulic gradient. In such a case the discharge of the portion of the pipe AC would have to be computed as taking place under the head h, which would be the difference in level between the water surface in the reservoir and the center of the pipe at C. From C to G, owing to the steepness of the grade, the pipe would only be partially full. In other words, the discharging capacity of the pipe from A to C would not be as great as from C to G.

When a pipe conveys water from a reservoir to another, having its water surface at a lower level, the total head acting on the pipe will be the difference in the elevations of the surfaces of the two reservoirs.

PART II

DESIGN AND CONSTRUCTION

CHAPTER IV

WOODEN PIPES

39. Water Pipes. — The earliest pipes for conveying water consisted of stone blocks through which cylindrical holes were cut. Earthen pipes, generally encased in masonry, were also used. Lead pipes were extensively employed by the Romans for distributing water, and, also, as inverted siphons for crossing valleys. In the Middle Ages bored logs or lead pipes were used for conveying water.

Modern water pipes are made of cast iron, wrought iron, steel, lead, wood, vitrified clay, cement or concrete. In eastern countries, bamboos are largely used for water pipes of small diameter. Descriptions of the different kinds of modern water pipes are given in the following pages.

40. Log Pipes. — About a hundred years ago, the pipes laid for distributing public water supplies consisted of logs of spruce, yellow

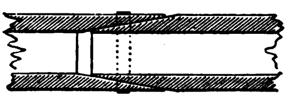


FIG. 20.

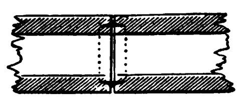


FIG. 21.

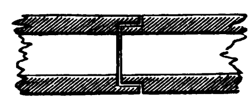


FIG. 22.

Log Pipes.

pine, oak, tamarack, about 12 ft. long and 12 to 15 ins. in diameter, which were bored out to diameters of 2 to 6 ins. The bark was generally, but not always, removed before the logs were laid. Originally the logs were bored by hand, but later machinery was invented for this purpose. Figs. 20, 21 and 22, taken from Charles S. Storrow's "Treatise on Water-works," Boston, 1835, show different manners in which the pipes were joined, the most approved method being by means of an inner iron ferrule or thimble (Fig. 21). The bands placed at the ends, to prevent splitting, were made of bar iron, about $1\frac{1}{2}$ by $\frac{3}{8}$ in. in size.

If placed in ordinary soil, and deep enough to be protected against atmospheric influence, log pipes will last a very long time. Pipes of this kind, which have been dug up in recent years in New York, Philadelphia, Boston, Detroit, after having been over a hundred years in the ground, have generally been found to be perfectly sound. Some of these log pipes taken up in Boston and Portsmouth, N. H., are said to be over two hundred years old.* Exeter, N. H., was supplied prior to 1886, by a wood pipe aqueduct for nearly a hundred years. In dry porous soil (gravel, sand, etc.), the wood is apt to decay, unless it is kept constantly saturated.

The first log pipes in America were laid in 1652 for the water works of Boston, Mass. They were replaced by new log pipes about 1796, and some of these pipes remained in service until about 1848, when they were all replaced by cast iron pipes. In New York and in Philadelphia the first log pipes were laid, respectively, in 1774 and 1799. Detroit, Mich., had 130 miles of small wooden water pipes. They were generally made of tamarack, and bored for a diameter of only $2\frac{1}{4}$ ins. In 1884, 78 miles of these log pipes were still in use in that city. Log pipes were

FIG. 23. — Wyckoff Wood-stave Pipe.

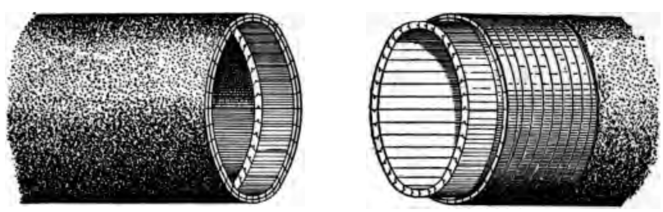


FIG. 24. — Joints of Wyckoff Pipe.

laid in Wilmington, Del., about 1816, and some of them are still in the ground.

After the great fire that destroyed part of London, England, in 1807, log pipes were laid to replace the lead pipes, as the latter were found to be melted by the heat of the great conflagration. By means of a special machine, a number of pipes of different diameters were obtained from the same log, the large core, cut out for the biggest pipe, serving as a log for the second largest pipe, etc. At one time there were more than 400 miles of these log pipes in London.

41. Wyckoff Water Pipe. — An improved kind of wooden water pipe was invented in 1855 by A. Wyckoff, of Elmira, N. Y. It consists of a wooden shell, bored and turned from a solid log, and strengthened

* Jour. New England Water Works Assoc., 1916, XXX, p. 318.

by a band of iron, steel or bronze, wound spirally around the pipe (Figs. 23 and 24). The outside of the pipe is protected against corrosion by an asphalt coating, about $\frac{1}{8}$ in. thick. Mr. Wyckoff obtained in 1855 a patent for an auger which bored a log in such a manner that a number of concentric pipe shells could be obtained from the same log. At first these pipes were principally sold in the mining districts of Pennsylvania, where pipes of iron or steel were quickly corroded by the acids contained in the waters of the mines, and they were only made to resist low pressures. By gradual improvements in the manufacture, especially by the invention of a machine with an automatic brake for applying the band under the required tension, these pipes are now made for bearing any pressure up to 172 lbs. per sq. in., and are used in the distributing system of some water works.

FIG. 25. — Cast Iron Specials for Wyckoff Pipe.

The Wyckoff pipes are now made in lengths up to 12 ft., with inner diameters of 2 to 6 ins. For low pressure, each length of pipe has a 3- or 4-in. tenon at one end, according to the size of the pipe, and a corresponding mortise at the other end. A tight joint is secured by driving the pipes together with a ram. For high pressures, some manufacturers bore a mortise at each end of the pipe, and the joints are made by means of wood thimbles, 8 inches long, which fit into the mortises. These thimbles, which are used only for pipes 2 to 4 ins. in diameter, are usually made of thoroughly seasoned white pine, and have a diameter $\frac{1}{8}$ in. larger than the diameter of the mortise.

The bands have widths of 1 to 2 ins., and various thicknesses, according to the pressure which is to be resisted. They are wound so as to have 0 to 3 ins. between the spiral wrappings, according to the strength required. The bands are usually made of No. 14, 16 or 18 gauge

Bessemer steel having a tensile strength of 70,000 lbs. per sq. in. For pipes which are to convey mineral water, containing large quantities of compounds of sulphur, the bands are made of Tobin bronze. The pipes intended for high pressure are all tested for 200 lbs. per sq. in., before leaving the shop. Iron specials (Fig. 25) are generally used with the Wyckoff pipe, but for low pressures the tees and crosses are made of wood (Fig. 26). Curved pipes are usually made by heading mortise and tenon on pipes slightly off center so that when two pipes are driven together they form a slight angle. Short pipes are generally used for this purpose, and by the use of from 4 to 6 short curves a right-angle turn can be accomplished.

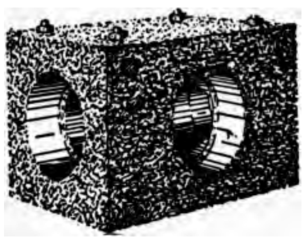


FIG. 26. — Wooden Special.

42. Machine-made Wood-stave Pipe. — The Wyckoff pipes, made of solid logs, were limited originally to sizes having inner diameters of about 10 ins. Owing to the difficulty of getting large logs, these solid pipes are now only made up to 6 ins. in diameter, and larger pipes are made of staves, somewhat like a barrel. Pipes of this kind have been used for penstocks since about 1850, but in all of these cases the staves were put together in the trench.

In 1860 Mr. Wyckoff began to make wood-stave pipes in his factory at Elmira, N. Y. The staves are of selected Canadian white pine, and are passed through a machine which planes the inner and outer faces to circular lines, and makes the edges radial. At the same time, the machine cuts a double groove, $\frac{1}{8}$ in. wide and $\frac{1}{8}$ in. deep, into one edge of each stave, and two tongues, $\frac{3}{16}$ in. wide by $\frac{3}{16}$ in. high, into the other edge. When the staves are banded together, the tongues, which are a trifle larger than the grooves, are squeezed into the latter, thus making water-tight joints. The tongues and grooves are cut $\frac{3}{8}$ in. from the outer and inner face of the stave. The pipes are banded with flat steel bands, wound spirally around the pipe, under sufficient tension to draw the staves closely together. The sizes of band used, and the spacing, are fixed by the pressure that is to be resisted. After the staves have been properly banded, one end of each pipe is recessed by a suitable machine, and the other end is turned into a conical tenon, which fits into the recess of the adjoining pipe. The pipes are united by simply driving them tightly together. The swelling of the wood at the joint, when saturated, makes the pipe perfectly water-tight.

As the life of a wooden pipe subjected to pressure will evidently depend upon that of the steel band, great pains have been taken by the manufacturers to coat the bands, as also the whole exterior surface of the pipe, with a durable water-proof, strongly adhesive material, which will not become soft in summer, nor brittle in winter. This coating, which is applied at a temperature of 250–300° Fahrenheit, consists of solid bitumen, produced by distilling Texas oil or the residuum of this oil, at the same time blowing air through the heated liquid. The oxidizing effect of the air turns the liquid residuum into a solid, which forms an excellent coating for the wooden pipes. While this coating is still soft, the pipes are rolled on to a table covered with sawdust, which adheres to the hot coating and prevents it from being knocked off or abraded during transportation.

43. Advantages of Wooden Pipes. — Wooden pipes have the following advantages, compared with metal pipes, which may often make it advisable to give them the preference:

1. They can be laid more cheaply, as they do not have to be leaded, caulked, etc.
2. They can be laid in a wet trench as well as in a dry one.
3. Their trench can be shallow, as wood, being a poor conductor of heat, is a protection against freezing.
4. Owing to the elasticity of the wood, there is less likelihood of bursting, on account of freezing or water-hammer.
5. They are more durable than iron pipes in alkaline soil.
6. As they do not become tuberculated, their discharge remains constant.
7. They are easily tapped for service pipes.
8. They are not liable to electrolysis.

For mains 12 ins. or more in diameter, wooden pipes, including the cost of laying, are considerably cheaper than those made of cast iron, wrought iron or steel. For small distributing pipes, 6 or 8 ins. in diameter, the difference in cost is not so great.

44. Tapping Wooden Mains. — Wooden mains are tapped in the following manner: A hole is bored in the wooden shell with an extension bit to within a quarter of an inch of the inside of the pipe. A brass corporation cock, having a long coarse thread, is screwed into the hole. The remaining quarter of an inch of wood is bored with a small bit, while the corporation cock is open. As the bit is withdrawn, the auger chips are forced out by the water pressure. An experienced workman will not lose a quart of water in making such a connection. In tapping wooden pipes, care must be taken not to damage the protective coating of the steel bands. The heavy coating of pitch is a good insulator against electrolysis. If electricity should reach the band at any point, on account of the pitch coating being abraded, it cannot go far, as there is an inch space between the bands at the adjoining ends of the pipes.

45. Wood-stave Pipes for Water Works. — As already stated, Wyckoff pipes were first used in mines. In 1860 such pipes were laid in the distributing system of the water works of Elmira, N. Y., under pressures of 35–86 lbs. per sq. in., and some are still in use in that city. In 1872 this type of pipe was adopted for the water works of Bay City, Mich., and, since then, it has been used in a number of water works in the Eastern and Middle States. The following table gives the diameters of the main pipes of some of these water works, and the heads to which they are subjected.

TABLE XXIII. — WOOD-STAVE PIPES IN WATER WORKS

Locality	Manufacturer	Diameter in inches	Head in feet
Jaffrey, N. H.	A. Wyckoff & Son Co.	30	200
Lee, Mass.		28	200
Windber, Pa.		24	250
Butler, Pa.		24	200
Akron, Ohio		20	170
Riley, Me.		16	200
Elmira, N. Y.		16	170
Canajoharie, N. Y.		12	270
Swanton, Vt.		12	270
Troy, N. H.	Michigan Pipe Co.	12	310
Houtzdale, Pa.		8	300

Machine-banded wood-pipe has been used extensively for municipal water works in the Pacific Coast and Rocky Mountain States, where most towns of consequence, at some time, used more or less of this kind of pipe. It has, also, been used extensively for conveying water for manufacturing purposes, for fire protection, and for power plants, etc.

46. The Michigan Pipe Co., Bay City, Mich., has manufactured wooden pipes since 1869. In 1880 this company acquired the right to manufacture Wyckoff pipes. It now makes pipes, 2 to 48 ins. in diameter and 8 ft. long, of well-seasoned white pine or tamarack. The pipes are wound spirally under heavy tension by machinery, either with galvanized steel wire or with flat bands. The wire-wound pipes are used for heads up to 350 ft.

47. Manufacturers of Machine-banded, Wood-stave Pipe. — Besides the two companies mentioned above, the following corporations are engaged in the manufacture of machine-banded wood-stave pipe: The Standard Wood Pipe Co., Williamsport, Pa.; The Pacific Coast Pipe Co., Seattle, Wash.; The Pacific Tank & Pipe Co., San Francisco, Calif.; The Washington Pipe Foundry Co., Tacoma, Wash., and the Portland Wood Pipe Co., Portland, Ore., and Vancouver, B. C.

The western manufacturers usually band their pipes with wire, instead of using flat bands, owing to the high price they have to pay for such bands. While the wire corrodes less rapidly than the bands, it cuts deeper into the wood under great pressure, and it is apt to cause leakage at the longitudinal joints, when the pressure is reduced. Some of the manufacturers provide a tenon on each side of the pipe, and make the joints by means of sleeves. These joints are said to be stronger than the tenon and mortise joint, described above; but it is difficult to coat the edges of the sleeves properly, and to keep them completely saturated by the water in the pipe.

48. Laying Machine-made Wood-stave Pipe. — In transporting the pipes from the mill to the trenches into which they are to be laid, care must be taken not to bruise the pipes or to injure the tenons and mortises. The pipes should not be exposed for any length of time to the sun or wind, so as not to shrink or be warped. This is especially apt to occur in pipes that are made in moist regions and transported to arid or semi-arid districts.*

Four to eight men are required for laying the pipes. The latter number, provided with ropes, can easily lower a 48-in. wooden pipe into the trench, without the use of a derrick.

TABLE XXIV. — WYCKOFF WOOD-STAVE PIPE†

Size of pipe in inches	Outside diameter of pipe in inches	Weight per lineal foot in lbs., 80 lbs. pressure	No. feet in carload, 40-ft. car	No. feet 2-horse truck can haul	No. men used in laying exclusive of foreman	No. feet these men can lay in day of 10 hours
6	$10\frac{1}{8}$	15	2700	344	4	2000
8	$12\frac{1}{8}$	17	2300	272	4	1900
10	$14\frac{1}{8}$	23	1800	216	4	1700
12	$16\frac{1}{8}$	25	1500	192	6	1500
14	$18\frac{1}{8}$	31	1100	152	6	1400
16	$20\frac{1}{8}$	33	900	144	6	1200
18	$22\frac{1}{8}$	38	750	128	6	1000
20	$24\frac{1}{8}$	42	500	112	6	800
24	$28\frac{1}{8}$	55	330	80	8	600
30	$36\frac{1}{8}$	90	225	64	8	450
36	$42\frac{1}{8}$	120	160	48	8	300
48	$54\frac{1}{8}$	150	95	24	8	200

† This pipe is now manufactured by A. Wyckoff & Son Co., Elmira, N. Y., and by the Michigan Pipe Co., Bay City, Mich.

Before joining the pipe to the last one laid, it should be carefully inspected, and, if any slight injury is discovered, the pipe should be turned so that the defective part comes on top, where it can be readily repaired, if necessary. The joints are made by simply driving the pipes together, a maul being used for pipes up to four inches in diameter, and a ram, made out of a square piece of timber, for larger sizes. A wooden

* Bulletin No. 155 of the U. S. Dept. of Agriculture, p. 28.

plug (tampion), strengthened by iron rings, is used to protect the end of the pipe, and the driving is usually done from the coupling or mortise end. Deflections of 2 to 6 degrees per joint can be made with this kind of pipe. On curves, short sections of pipe should be used. The back filling should be carefully tamped or puddled, especially on curves, to prevent blowing out under pressure. Metal bends or plugs should be well anchored.

The weights of machine-made wood-stave pipes, 6 to 48 ins. in diameter for a pressure of 80 lbs., and the cost of laying them, is given by the Wyckoff Company in Table XXIV.

The cost of laying machine-made wood-stave pipe, not including distribution along the trench, excavation and back filling, is estimated by different manufacturers of pipe as follows:

TABLE XXV. — COST OF LAYING WOOD-STAVE PIPE
(Cents per lineal foot)

Manufacturer	Diameter of pipe in inches													
	4	5	6	8	10	12	14	16	18	20	24	30	36	48
Wyckoff Co.	1	1	1	2	3	3	4	4	4	5	6	7	8	10
Michigan Co.	1	1	1	2	3	3	4	4	4	5	6	7	8	10
Pacific Coast Co.	1½	...	...	...	...	...	...	...	...	...	5	...	...	...
Portland Co.	1	...	1½	1½	2	2½	...	...	...	...	...	...	...	...

49. Continuous Wood-stave Pipes. — Wood-stave pipes were first used for penstocks for water-wheels. A pipe of this kind was built by B. H. Hull * as early as 1851, in 12-ft. lengths, of 2-in. pine planks, which were tapered, so that the sections could be fitted together in stove-pipe fashion. The pipe was banded with solid $\frac{1}{4}\times$2-in. wrought iron bars. In later pipes, which were 6 to 10 ft. in diameter, Mr. Hull made the staves break joints, to make the pipes more water-tight, and he used $\frac{3}{8}\times$2-in. wrought iron bars having screw ends, which passed through cast iron shoes.

In 1874 J. T. Fanning, M. Am. Soc. C. E., built a wood-stave pipe, 6 ft. in diameter and 600 ft. long, for a penstock at Manchester, N. H.† It was made of 4-in. southern pine staves with radial edges. The hoops were of wrought iron, 2½ ins. wide and ¼ to ½ in. thick, according to the pressure. Each band was made in two pieces, and had two tightening bolts, opposite to each other. The staves met at their ends in butt joints, which were made tight by inserting in them small pieces of hoop-

* *Eng. News*, June 20, 1891, p. 595.

† Trans. Am. Soc. C. E., 1877, VI, p. 69.

iron, $\frac{3}{4}$ in. wide and $\frac{1}{8}$ in. longer than the width of the staves. These pieces of iron were placed in *saw-kerfs* which were cut in the staves. The ends of the iron plates were sharpened so as to indent the sides of the adjoining stave. This pipe was put into use on July 4, 1874, the maximum head of water to which it is subjected being 40 ft. It remained in service until the fall of 1912 when it was replaced by a steel pipe.

The City of Toronto, Canada, built a wood-stave pipe, 4 ft. in diameter and 7000 ft. long, to carry water for its domestic supply from a filtering gallery on an island in Lake Ontario to a pumping station on the mainland.

Denver, Col., is the first place where wood-stave pipes were used extensively as water-mains. In constructing in 1884 a pipe-line to bring a new water-supply to this city, C. P. Allen, M. Am. Soc. C. E., Chief Engineer of the Citizens Water Company of Denver, decided to build a 48-in. wood-stave pipe for this purpose, as the alkaline soil of Colorado was known to corrode wrought iron or steel pipes very rapidly, and as the cost of cast iron was prohibitory, Mr. Allen took out, in 1887, a patent for the design of wood-stave pipes, adopted by him. Other pipe-lines were afterwards built under these patents for the Citizens Water Company of Denver, which now owns about 100 miles of wood-stave pipes, some of which have been in the ground for over 20 years. Wood-stave pipes were built in other places, both for water supply and for water power works.

In 1909 a continuous wood-stave pipe, 48 ins. in diameter and 25,500 ft. long, was built across a salt marsh to supply Atlantic City, N. J., with water.* Several lines of metal pipes, which had been previously laid, had been failures.

A number of large continuous wood-stave pipes have been built of California redwood or Douglas fir, known also as Oregon or Washington yellow fir, in connection with water power developments.† The Pacific Coast Pipe Co., of Seattle, Wash., built in 1913 for the North-western

* Jour. Am. Soc. Engineering Contractors, 1912, IV, p. 301.

† Fir is a bastard spruce, first named by Douglas. It is the most common wood of the Pacific Northwest, and is known locally as Douglas fir. A distinction is sometimes made between yellow and red fir, but both come from the same kind of tree, the former, which is more common, being cut from large trees, while the latter is obtained from small trees. The color is somewhat affected by exposure to the sun. In California these trees are called Oregon fir. The wood is strong, contains pitch in seams and pockets, and can be obtained in pieces averaging 20 ft. in length.

Redwood grows on the Pacific Coast from Santa Cruz north into southern Oregon. Owing to the large diameters of the trees, the logs are cut in shorter lengths than fir, the average length of the commercial timber being about 16 ft. The wood is not as strong as fir.

Electric Co., of Portland, Ore., a continuous wood-stave pipe, $13\frac{1}{2}$ feet in diameter and about a mile long, which is the largest pipe of this kind thus far constructed.* Throughout its whole length the pipe is laid on timber cradles, spaced about $4\frac{1}{2}$ ft. from center to center.

The same company built a large wood-stave pipe near Altmar, N. Y., for a subsidiary of the Ontario Power Co., of Niagara Falls, N. Y. For 3450 ft. this pipe has a diameter of 12 ft., and for 4400 ft. the diameter is 11 ft. The 12-ft. pipe was thoroughly coated on the outside with avenarius carbolineum, as a precaution against decay spores getting a start, during seasons of low water, when there might not be sufficient head available to saturate the wood thoroughly.

The Pacific Tank and Pipe Co. is another western corporation engaged extensively in the manufacture of wood-stave pipes.

Wood-stave pipes can be used for crossing valleys as inverted siphons, providing they are not exposed to a greater head than about 200–300 ft. They have been used in this manner to carry the water of irrigation canals across valleys. The West Side Irrigation and Mining Canal in Wyoming and Colorado is carried across three valleys by wooden siphons, 3 ft. in diameter.† The canal of the Rio Grande Dam and Irrigation Company is continued across the Rio Grande River by a siphon consisting of four lines of 50-in. wood-stave pipe.‡

Another use for which stave pipes are well adapted is as the outfall of sewers. Pipes of this kind, built of creosoted wood, have been used for this purpose in New York, Brooklyn, and other places. The pipe is constructed in 20-ft. lengths, which are floated into position and then sunk. A mouthpiece of galvanized iron is provided. The pipe may be given a circular or elliptical cross-section.

50. Construction of Continuous Wood-stave Pipes. — Great care must be exercised in the selection of the timber for continuous stave-pipes. The staves are milled out of commercial sizes of scantling ($1\frac{1}{2}$ to 3 ins. thick). Only live wood, free from pitch, rotten knots, shakes, cracks and other imperfections should be used for this purpose, and it should be thoroughly seasoned before being milled into staves. Kiln-dried lumber is generally used for these pipes. The staves should all be straight-grained and free from warp or wind. They should be strong enough to resist crushing under a firm tensile strain on the bands, and should not become spongy, when saturated. California redwood has been used largely for wood-stave pipe, but good results have also been obtained with various classes of pine, spruce and fir. Both sides are

* *Eng. Record*, Oct. 11, 1913, p. 396.
† *Eng. News*, July 30, 1896, p. 66.
‡ *Ibid.*, July 15, 1897, p. 34.

cut to the required curvature of the pipe, the thickness of the stave thus remaining uniform, and the edges are cut radially. A slight projection or bead, about $\frac{1}{8}$ in. high, is cut on one edge of each stave, and indents the edge of the adjoining stave, when the bands are tightened. It serves to offset slight irregularities that may occur in the milling. Leaks rarely occur in the longitudinal joints of the pipes.

The pipes are assembled in the trench. This is a great advantage as regards the transportation of the material. When the staves and bands are distributed, the building of the pipe may be commenced at as many points as the progress in trenching will permit. Gangs of 8 to 16 workmen, according to the number of bands to be placed, commence at points about 1000 to 2000 ft. apart, and work towards each other. The

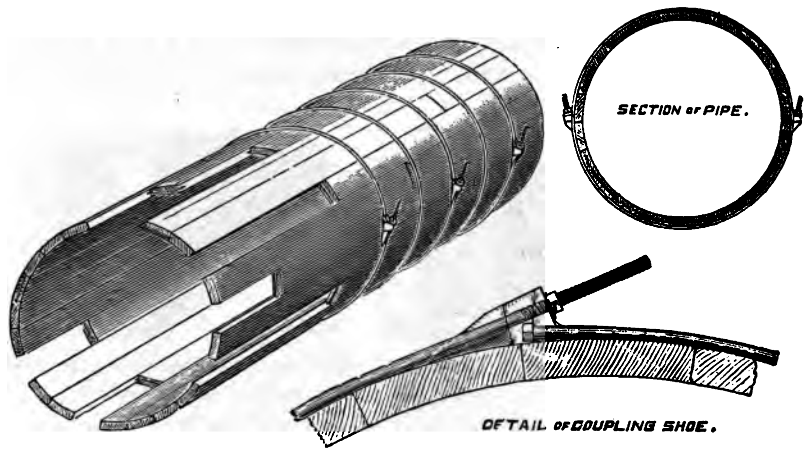


FIG. 27. — Continuous Wood-stave Pipe.

staves are of different lengths of about 14 to 16 ft. They are made to break joints in the pipe (Fig. 27). While there are, therefore, no circumferential joints in the pipes, the joining of the staves usually occurs within a space of 2 or 3 ft., where two or three extra bands are placed, if the water pressure is to be considerable.

The staves are butted together at their ends, the joints being staggered, so that they are never nearer than 6 ins. together. To insure tightness at the butt joints, metallic tongues are inserted, $\frac{3}{4}$ in. deep, in saw-kerfs in the ends of the staves (Fig. 28). These tongues are made long enough to project $\frac{1}{8}$ in. in either side of the stave, so as to indent the adjoining stave when the bands are tightened. By this simple arrange-

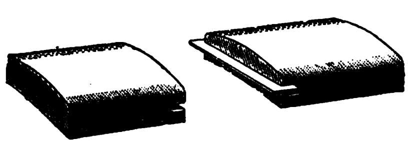


FIG. 28. — Saw-kerf.

ment, which was already used in 1874 in the Manchester pipe, the leakage at the butt joints is prevented. The *saw-kerfs* must be cut tangentially to the curve of the pipe and with great accuracy.

The bands are usually made of a mild steel * and have a circular or elliptical section. They are made 4 to 5 ins. longer than the perimeter of the pipe, to facilitate rapid work in placing them. One end of each band has a head, and the other is upset and has a thread, about 5 ins. long. The bands, which are shipped as straight pieces, are bent on the ground around circular tables, and are painted or dipped into a hot bath of asphalt. The shoes (Figs. 29 and 30) are made of the best tough cast iron, or of the best quality of malleable iron, and are coated with the same paint or asphaltum used for protecting the bands.

In constructing the pipes, the bottom staves are placed inside of U-shaped molds, made generally of $1\frac{1}{2}$-in. gas pipe. Centers, formed

FIG. 29. — Coupling-shoe for Wood-stave Pipe.

FIG. 30. — Coupling-shoe for Wood-stave Pipe.

of the same kind of gas pipe, are set up for the upper staves. Where the different sections meet, the last staves are cut $\frac{1}{8}$ in. too long, and are sprung into position. During the placing and tightening of the bands, the staves are driven tight at their end joints, and the inside of the pipe is rounded out (coopered). The bands must be screwed up (cinched) sufficiently tight to produce a uniform contact between the band and all the staves, and to cause a lateral compression between the staves, which will not be entirely neutralized, when the pipe is subjected to the full water pressure. It is impracticable to secure these objects, without causing undue strains in the bands, by simply screwing the ends of the band. It is found necessary to hammer the band at all points during the cinching, to make it indent the wood uniformly and sufficiently, to secure the required bearing surface between the wood and the iron. If this surface is not as large as experience shows

* Steel having a tensile strength of 58,000–65,000 lbs. per square inch, an elastic limit of 40,000–50,000 lbs. and an elongation of 24–27 per cent in 8 ins.

to be necessary, the fiber of the wood will evidently be crushed under the band.

When the bands are only two or three inches apart, great care must be taken in the cinching, in order to avoid crushing the wood or collapsing the pipe. If this occurs, serious leakage will take place, when the pipe is subjected to the full water pressure. The final cinching of the pipe is usually deferred until the day after the pipe is laid. In order to avoid screwing the rods too much, brace wrenches having not over ten inches leverage are used. In order to obtain good results, continuous wood-stave pipe should be made and laid by contractors who have had experience in this kind of work.

The amount of work that can be accomplished per day in building stave pipes depends upon the size of the pipe and the closeness of the bands. James D. Schuyler, M. Am. Soc. C. E., states that on a 34-in. pipe in Denver each gang of workmen built 150–300 lineal ft. of pipe per day. He gives the cost of the erection of the pipe as 5 to 16 cents per band for 30 to 48 in. pipes. As a rule, wood-stave pipes are placed in trenches, and covered with earth of sufficient depth to protect them against the action of the elements, and against injury from falling rocks or trees, or from fire. Sometimes only the lower half of the pipe is covered with earth, but this plan is not to be recommended, as decay is apt to set in. In some cases, especially where the pipes are very large, they have been laid on the surface on wooden or masonry cradles, spaced about 4 ft. apart, and have been carried across depressions on bridges or trestles. As a rule, exposed wooden pipes will not last as long as covered pipes, but they are more easily inspected and repaired.

One of the great advantages possessed by stave-pipes is that they can be built in the trench to follow any ordinary curve. Pipes of 40 to 48 ins. in diameter can be readily constructed on curves of 250 ft. radius. When sharp turns are to be made with stave-pipes, special castings are required for the angle points.

51. The Design of Wood-stave Pipes.* — The staves must have sufficient thickness to prevent percolation, to resist the lateral compression to which they are subjected, and to insure that the pipe, when empty, will not collapse under the load of the refilling. At the same time, they must be thin enough to become perfectly saturated, and to bend easily, where the pipe has to be laid on a curve. The thickness of the staves depends, therefore, upon the diameter of the pipe, and upon the maximum pressure to which it is subjected. It is determined mainly by experience.

* Trans. Am. Soc. C. E., 1899, XLI, p. 27.

Robert E. Horton, M. Am. Soc. C. E., gives the following empirical rule for determining the thickness of the staves:*

Thickness of stave in ins.:

$$1 \text{ in.} + \frac{\text{head in ft.}}{100} + \frac{\text{diam. of pipe in ins.}}{100}. \qquad (30)$$

In actual practice the thickness of the staves may vary from $\frac{1}{8}$ to $\frac{1}{4}$ in. from that given by the rule.

For the smallest radius to which continuous stave-pipes can be laid on curves, Mr. Horton gives the following approximate rule:

$$\text{Minimum radius in ft.} = 4 \text{ to } 5\ (D + 4\,t^2), \qquad (31)$$

where D = diameter of pipe in inches;
t = thickness of staves in inches.

In order to prevent the pipe from being deformed by its weight and that of the water it contains, it should be supported through an arc extending about 55 degrees on each side of the vertical axis.

52. The spacing of the bands must be such that the stress caused in each band by the water pressure and the swelling of the wood will not exceed a safe limit, assumed usually for mild steel at about 16,000 lbs. per sq. in. The spacing of the metal bands is determined by the following formulas:†

Let r = internal radius of the pipe;
t = thickness of staves in ins.;
d = distance between centers of bands in ins.;
s = tensile strain in the band in lbs.;
e = permanent swelling force of the wood in lbs. per lineal in. of band;
p = pressure of water in lbs. per sq. in.;
b = safe bearing power of the wood in lbs. per lineal in. of band;
c = average compressive lateral strain in lbs. per sq. in., when the pipe is under pressure of p lbs.

The strain produced in the bands by the water-pressure is readily calculated. The greatest pressure which the band has to sustain is evidently $2\,rpd$. As this is resisted by the tension in the band at two points diagonally opposite each other, we have

$$s = rpd. \qquad (32)$$

* Report by Robert E. Horton on "Continuous Stave Pipe," published by the Marion Malleable Iron Works, Marion, Ind., in its booklet on "Malleable Iron Shoes for Continuous Stave Pipes."

† These formulas are given by H. C. Henny, M. Am. Soc. C. E., who, as manager of the Excelsior Wooden Pipe Co., of San Francisco, Cal., had large experience in the construction of continuous wood-stave pipes. (Trans. Am. Soc. C. E., 1899, XLI, p. 68.)

To insure water-tightness, the staves must be under some compression, when the pipe is under pressure. This is secured by proper *cinching* before the water is admitted. As the compression in the staves produces an additional strain in the bands, we have, for the whole strain, the formula:

$$s = rpd + ctd. \tag{33}$$

If the pipe has not been cinched so tight that the wood cannot swell laterally, c, in the above formula, will be increased by the swelling of the wood when it becomes saturated. As wood in this condition will evidently yield, if subjected to a greater compression per square inch than the force it exerts in swelling (just as a spring would yield if put under a greater pressure than the force it exerts), the maximum value of c for saturated wood is evidently equal to e, and we have, therefore, for a saturated pipe:

$$s = prd + etd. \tag{34}$$

In order that the wood under the pipe should not be crushed, d must be given such a value that the strain in the wood under the band will not exceed b. Considering b to be produced by a uniform exterior pressure, just as s was calculated for an interior pressure, we have the following equation of condition,

$$s_{\text{max.}} = rpd + etd \leqq (r + t)\, b. \tag{35}$$

Formula (34) gives a greater strain in the bands than is necessary to produce tightness. If the nut of a band is turned while the pipe is under pressure, the compression in the staves will be reduced, and finally leakage will take place at points about halfway between the bands, where the compression in the staves is least.

If c_1 and c_2 represent, respectively, the maximum and minimum compression per square inch in the stave, *viz.*, that occurring at the bands and halfway between them, the average compression will be approximately:

$$\frac{c_1 + c_2}{2}. \tag{36}$$

In order to insure water-tightness, we should have $c_1 > p$, as otherwise the staves would be forced apart if the water should get between them.

If the bands are not spaced too far apart, and the staves are not too thin, c_1 will not be much greater than c_2. Under these conditions it is believed that water-tightness is insured by the condition

$$c_1 + c_2 = 3\,p, \text{ or } c = \tfrac{3}{2}\,p. \tag{37}$$

Substituting this value in formula (33) we have for the least band strain in a tight pipe:

$$s = prd + \tfrac{3}{2}\,ptd, \tag{38}$$

which should be equal to the safe tensile strain in the band. To limit the pressure on the wood under the band to b we must have

$$s = pd\,(r + \tfrac{3}{2}\,t) \leqq (r + t)\,b, \tag{39}$$

from which we obtain:

$$d = \frac{s}{p\,(r + \frac{3}{2}\,t)} \leqq \frac{(r + t)\,b}{p\,(r + \frac{3}{2}\,t)}. \tag{40}$$

53. Horton's Formulas. — Simple formulas for determining the proper size and spacing of the bands are given by Robert E. Horton in his report on "Continuous Stave Pipes." The formula for finding the required size of band is as follows:

$$a = \frac{(pd + 200\,t)}{2\,s}\,f, \tag{41}$$

where a = required area of band section per inch length of pipe;
p = pressure of water in lbs. per sq. in.;
t = thickness of staves in inches;
s = ultimate tensile strength of bands per square inch;
f = factor of safety.

s is usually taken as 60,000 lbs. and f is assumed as 4 for exposed pipes, increasing to 5 for pipes subject to water-hammer, or for pipes buried in the earth.

When the bands are cinched with the wood dry, the resistance of the wood to compression exerts, according to Mr. Horton, a stress on the bands which may be as great as 1650 to 2000 lbs. per sq. in. of edge surface of the staves. This pressure is greatly reduced as soon as the pipe is filled with water, as the water pressure expands the bands and consequently diminishes the compression on the edges of the staves. On the other hand, the swelling of the wood increases the compression in the stave edges again. The net pressure per square inch in the edges of the staves, due to the forces mentioned, is usually assumed at 100 lbs. per sq. in. of edge surface of the staves, and this figure is used in the above formula.

In order to avoid dangerous crushing of the wood, the pressure between the bands and the wood should not exceed about 800 lbs. per square inch of contact. For round bands the width of contact between the band and stave should be about equal to the radius of the band. Adopting these data, Mr. Horton gives the following formula for determining the maximum permissible spacing between bands of given size under a given pressure:

$$S = \frac{400\,d}{p}, \tag{42}$$

where S = maximum permissible spacing of bands in ins.;
d = diameter of round bands in ins.;
p = water pressure per sq. in.

For the lightest pressures, the spacing of the bands is usually made 10 to 12 ins., and for greater pressures, this distance must be correspondingly reduced. Different sizes of bands and spacings may, evidently, be adopted for the same given diameter of pipe and head to which it is exposed. Small bands, spaced closely together, are apt to insure water-tightness; but, on the other hand, they are not as durable as larger bands. Practically, the nearest commercial size of band is selected. Having decided upon the size of band to be used in a given case, the spacing of the bands will be determined by the following formula:

$$S = \frac{A}{a}, \tag{43}$$

where S = distance between band centers in ins.;
A = area of band section in sq. ins.;
a = required band area per lineal inch of pipe.

TABLE XXVI. — ECONOMIC PROPORTIONS FOR CONTINUOUS WOOD-STAVE PIPE DESIGN

Nominal diam. of pipe, inches	Stock size for staves, inches	Thickness of finished staves, inches	Economic size of bands, inches	Working stress in band, pounds	Factor of safety in band
			Oval		
10	$1\frac{1}{2} \times 4$	$1\frac{1}{16}$	$\frac{5}{16} \times \frac{7}{16}$	1250	5.3
12	$1\frac{1}{2} \times 4$	$1\frac{1}{8}$	$\frac{5}{16} \times \frac{7}{16}$	1470	4.5
14	$1\frac{1}{2} \times 4$	$1\frac{3}{16}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4.0
16	2×6	$1\frac{7}{32}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4.0
18	2×6	$1\frac{3}{8}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4.0
20	2×6	$1\frac{3}{8}$	$\frac{5}{16} \times \frac{7}{16}$	1650	4.0
			Circular		
22	2×6	$1\frac{3}{8}$	$\frac{3}{8}$	1510	4.5
24	2×6	$1\frac{3}{8}$	$\frac{3}{8}$	1650	4.0
27	2×6	$1\frac{7}{16}$	$\frac{3}{8}$	1650	4.0
30	2×6	$1\frac{1}{2}$	$\frac{1}{2}$	2670	4.5
36	2×6	$1\frac{9}{16}$	$\frac{1}{2}$	2950	4.0
42	2×6	$1\frac{5}{8}$	$\frac{1}{2}$	2950	4.0
48	2×6	$1\frac{11}{16}$	$\frac{1}{2}$	2950	4.0
54	$2\frac{1}{2} \times 8$	$2\frac{1}{8}$	$\frac{5}{8}$	4600	4.0
60	3×8	$2\frac{1}{2}$	$\frac{5}{8}$	4600	4.0
66	3×8	$2\frac{9}{16}$	$\frac{3}{4}$	6600	4.0
72	3×8	$2\frac{5}{8}$	$\frac{3}{4}$	6600	4.0

Table XXVI gives the sizes of bands and spacings recommended by A. L. Adams for continuous wood-stave pipes, 10 to 72 ins. in diameter.*

The flow through wooden pipes is not diminished, in course of time, as is the case with iron and steel pipes, whose areas are reduced by tuberculation and corrosion.

* Trans. Am. Soc. C. E., 1899, XLI, p. 76.

CHAPTER V

CAST IRON PIPE

54. Historical Notes. — The precise date of the introduction of cast iron is unknown, but this material appears to have been used in the fourteenth and fifteenth centuries. Castings of iron made in the latter date in Sussex, England, are said to be still extant.* In the sixteenth century cannons, some weighing 3 tons, were made of cast iron, and the manufacture of cast iron water mains was, probably, begun about this time.

In 1664 to 1688 cast iron pipes, about 13 to 20 ins. in diameter, were laid, by order of Louis XIV of France, to supply the town and parks of Versailles from the reservoirs of Picardie.† They are 1 meter in length and are joined by means of bolted flanges. Some of these early pipe-lines are still in use. In 1901 some of these pipes were taken up after being more than 200 years in the ground, and were found to be of good gray iron, clean on the inside, and but little rusted on the outside. The rust appears to have formed a protective coating which prevented the penetration of oxygen beyond the surface of the metal. Ideal soil conditions and good water contributed to its long life.

In 1746 the Chelsea Water Co., of London, laid a 12-in. cast iron flanged pipe, which had to be relaid in 1791 on account of defective joints.‡ Thomas Simpson, the engineer of this company, designed, about 1785, the first bell-and-spigot pipe with lead joints. This kind of pipe proved to be so satisfactory that it was soon adopted by all of the London water companies.

In the United States all water pipes were made of wood up to 1817, when some cast iron pipes, imported from England, were laid in Philadelphia.§ These pipes were in lengths of 9 ft. and had bell-and-spigot joints. Some of them, which have been about 100 years in the ground, are still in use.¶ In the City of New York some uncoated cast iron pipes, which were imported from Scotland for the first Croton water works, are still in service. The earliest cast iron pipes were about $2\frac{1}{2}$ ft.

* "Encyclopædia Britannica," 9th Edition, Vol. XIII, p. 297.

† Jour. of New England Water Works Assoc., 1904, XVIII, p. 218.

‡ "Pipe and the Public Welfare," by R. C. McWane, p. 16, New York, 1917.

§ *Ibid.*, p. 17.

¶ Trans. Am. Soc. C. E., 1914, LXXVIII, p. 810.

in length, and were provided with flanged joints and leather gaskets which were bolted together. At a later date, somewhat longer pipes with screw joints were used, but trouble resulted on account of the rigidity of the joints, which prevented expansion and contraction. Next, cylindrical socket-joints were tried. They were turned to a conical form, and were driven together with a little whiting and tallow for luting. Later, the length of pipe was increased to about 9 ft., and the bell-and-spigot joints were used, being first adapted to a packing of wooden wedges, and later to a packing of lead. This form of joint, with slight modifications, continues to be used in both the United States and abroad. The length of pipe has, however, been increased, and cast iron pipes are now made so as to have an effective length of 16 ft., exclusive of the bell.

The first cast iron pipes were not coated with any preservative composition, although they were sometimes given a wash of lime. Uncoated pipes are still used for gas mains. Rust penetrates the metal on the inside and outside only for a slight depth, and then preserves the pipe from further corrosion. When a main has, however, to convey water, its capacity is gradually reduced by tuberculation (p. 461) and for this reason water pipes are now always protected by some kind of coating.

In 1848 Dr. Angus Smith patented in England a cheap and efficient coating for protecting cast iron from corrosion. It consists of gas-tar, pitch, linseed oil and resin, and with some improvements in the method of applying it, is now almost universally used for protecting pipes.

55. Thickness of Cast Iron Pipes. — The cast iron of which water pipes is made should resist safely a tensile strength of 16,000 to 18,000 lbs. per sq. in. In computing the thickness of metal required in a cast iron pipe to resist a certain water pressure, an allowance must be made for possible water-hammer, depending upon the diameter of the pipe. An additional allowance — about $\frac{1}{10}$ of an inch — must be made for the possibility of the core's being placed slightly eccentric when the pipe is cast, thus causing an inequality in the thickness of the metal. Beside theoretical considerations, the thickness of cast iron pipe must evidently be sufficient to prevent the breaking of the pipe in handling, and to resist corrosion. The least thickness which should be given to a small cast iron pipe should not be less than $\frac{3}{8}$ of an inch.

A large number of empirical formulas for computing the thickness of metal of cast iron pipe has been proposed. Due allowance is made in all of them for the practical considerations mentioned above.

James B. Francis's formula, which is used by the Warren Foundry and Machine Co., is:

$$t = 0.000058\,hd + 0.0152\,d + 0.312, \qquad (44)$$

where h = head in feet;
d = diameter of pipe in inches.

J. T. Fanning gives the following formula:

$$t = \frac{(p + 100)\,d}{0.4\,s} + 0.333\,\frac{(1 - d)}{100}, \qquad (45)$$

where t = thickness of pipe in inches;
p = pressure of water per sq. in.;
s = tensile strength of iron per square inch.

The formula used for the Metropolitan Water Works of Boston is:

$$t = \frac{(p + p')\,r}{3300} + 0.25, \qquad (46)$$

where t = thickness in ins.;
p = static pressure in lbs. per sq. in.;
p' = allowance for water-hammer in lbs. per sq. in.;
r = radius of pipe in ins.

In this formula 0.25 is the allowance for eccentricity, deterioration and safety in handling. The tensile strength of cast iron pipe is assumed to be 16,500 lbs. per sq. in., and a factor of safety of 5 is provided.

The value of p' in the formula is assumed as follows:

Size of pipe, inches	Value of p'	Size of pipe, inches	Value of p'
3–10	120	24	85
12	110	30	80
16	100	36	75
20	90	42–60	70

56. Special Castings. — Curved pipes, called, also, *bends*, and the castings required for uniting pipes at their intersections, for turning them to one side to avoid obstacles, for reducing or enlarging the diameter of a pipe-line, are usually classified as *special castings*, and cost about $2\frac{1}{2}$ to 7 times as much per pound as ordinary straight pipes.

57. Standard specifications for straight pipes and specials were adopted by the New England Water Works Association in 1902 and by the American Water Works Association in 1908. The specifications of the latter association, which are used very extensively throughout the United States, are given in the Appendix.

58. Pipe-joints. — There are three principal ways for joining cast iron pipes, *viz.*, by bell-and-spigot joints, flanged-joints, or by *turned-and-bored* joints. The first kind (Fig. 31) is almost universally used, on account of its making ample provision for expansion and contraction, and, also, on account of its flexibility and strength. The flexibility of the ordinary bell-and-spigot joint is shown by the facility with which a line of mains with such joints can be moved to one side, raised or lowered, within certain limits, without causing the joints to leak.

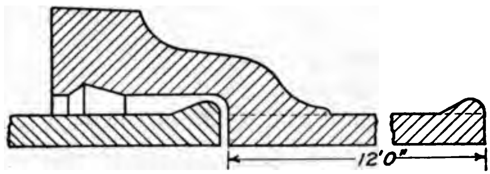


FIG. 31. — Bell-and-spigot Pipe-joint.

59. *Bell-and-spigot joints* should be designed to reduce the amount of lead used to the minimum permitting good caulking, and to make the bell sufficiently strong to withstand the stresses caused by caulking. Experience is the best guide in these matters. Wooden wedges have been occasionally adopted, instead of lead, to make a tight packing for bell and spigot. Such joints have been used at Yarmouth, Nova Scotia, since 1851, and have proved to be so durable, that they are now universally used in that place.*

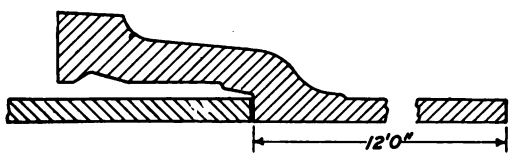


FIG. 32. — Dutch Pipe-joint with Plain Spigot.

Fig. 32 shows a Dutch type of joint which is used by the Public Works Department of The Hague, Netherlands. The bell is beveled and the pipe is without a spigot. It is claimed that this shape facilitates the centering of the plain ends in the beveled bottom of the bell.

60. *Flanged-joints* (Fig. 33) are advantageous for places where the pipes may have to be disconnected, such as at pumping stations, at water-gates, etc. They are easily made water-tight by covering the faces of the flanges with red-lead putty, in which a string that makes two or three turns is embedded, and then screwing the flanges together with bolts. Instead of the red-lead, sheets of rubber and cotton, corrugated copper, lead or copper wire may be used. Usually this kind of joint has no provision for expan-

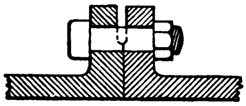


FIG. 33. — Flanged Pipe.

* "Water-works Engineering," by J. T. Turner and A. W. Brightmore, p. 340, London, 1893.

sion and contraction. Some allowance may be made, however, by placing an India rubber ring between the two flanges, as shown in Fig. 34, which is a joint which has proved effective in high-pressure mains in England.* Another way of making provision for expansion and contraction is by using a bell-and-spigot joint at every tenth joint. Bolted joints of the flange form should be avoided for underground work, as they are too rigid, and as the bolts will rust in time.

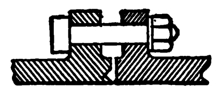


Fig. 34. — English Flanged Pipe.

61. *Turned-and-bored joints* (Fig. 35) involve some expense in the manufacturing, but for large orders this may be offset by the saving of lead, gaskets, etc. After the bored or turned parts are cleaned, they are coated with a solution of sal ammoniac or with a mixture of tallow or resin. The pipes are then simply pushed tightly together, the sal ammoniac, if used, making a rust joint. The principal objection to this kind of joint is that it makes no provision for changes of temperature, which

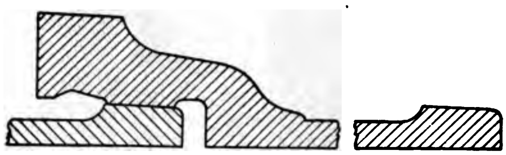


Fig. 35. — Bored Bell and Turned Spigot Pipe-joint.

may, therefore, cause the pipes to break or draw apart. Allowance for expansion and contraction for a pipe-line with bored and turned joints may be provided by making every tenth joint of the ordinary bell-and-spigot kind. Although bored and turned joints have, thus far, only been used to a very limited extent in the United States, they have been adopted in many foreign plants. Where the pipes can be laid in perfectly straight lines, this kind of joint has some great advantages. By greasing the joints, they become practically water-tight, independently of the lead which is always added.

62. *The taper joint of the universal pipe,* described on page 83, is another type of joint that has been used for cast iron pipe. The two joints that fit together have slightly different tapers. This gives the joint some flexibility.

63. Making Joints of Bell-and-spigot Pipes. — In making ordinary bell-and-spigot joints, sufficient hemp yarn should be wrapped around the spigot end, before it is placed in the bell. The hemp is then packed by a yarning tool; the joint is closed by a hemp-rope covered with plastic

* "Water-works Engineering," by Turner and Brightmore, p. 340, London, 1893.

clay (called a snake), an opening being left at the top, through which the lead can be poured. The lead is melted in a ladle, placed in a furnace, and it is important that the whole joint be poured in one operation, even for pipes as large as 48 ins. in diameter, requiring about 170 lbs. of lead for each joint. After the lead has been poured, the excess of lead is trimmed off, and the joint is caulked, generally by hand, by using three or four caulking tools. Machines for caulking are also in use (p. 475). For large pipes, a steel clip having a joint at the bottom is used, instead of the rope and clay. After the clips have been placed around the pipe, a small basin for receiving the molten lead is made with clay at the upper end of the clips.

In wet trenches it is dangerous to pour lead in joints. A little water in the joint, coming in contact with the molten lead, will cause an explosion.

FIG. 36.

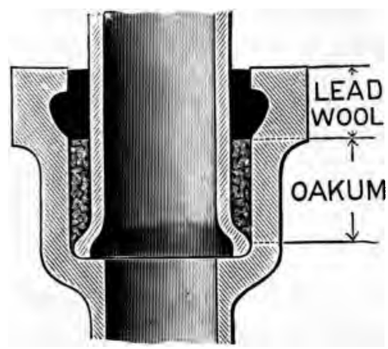


FIG. 37.

Lead Wool Joint.

This may be avoided, to a certain extent, by pouring a little kerosene or crude oil in the joint, before the lead is poured. For very wet trenches, the joints may be poured around a pipe in a dry place, the spigot end being then pulled out of the bell, and the lead joint being cut into halves. These pieces of lead can then be driven cold into a joint in the trench and caulked. A better material for such places is, however, the lead wool described below.

With a view of avoiding electrolysis, cement, mortar or wood, instead of lead, has been placed in bell-and-spigot joints, at certain intervals, along a pipe-line.

64. *Lead wool* * (Figs. 36 and 37) consists of finely divided lead in continuous strands in the form of rope. It is used, without heating, to fill the joints of water and gas mains, to repair leaks in pipes, masonry,

* Manufactured by United Lead Co., New York.

etc., and for other emergencies. The use of this material originated in Germany, where large cast iron water pipes are usually caulked with lead wool. In this country this material has been used for about ten years, especially for gas mains and for water pipes laid either in wet trenches or under water.

Lead wool is manufactured in several ways, the methods generally adopted being the extrusion, or a shaving process. Either of these processes produces very finely divided lead, which, when submitted to high pressures, unites and forms a very dense material, which has great pressure resisting qualities.

Lead wool is always placed cold in a pipe-joint. As each skein or strand is caulked separately, as it is introduced into the joint, the lead becomes much denser than when it is poured in the usual way. For this reason less depth of lead in a water main joint is needed with lead wool than with cast lead, which shrinks in cooling from the iron. In the latter case the subsequent caulking forces the lead out again only to a limited depth.

TABLE XXVII. — APPROXIMATE QUANTITIES OF LEAD WOOL AND YARN REQUIRED FOR C. I. PIPE-JOINTS

For Pressure up to 500 Lbs.

Lead wool			Yarn
Diam. of pipe	Depth, inches	Weight, lbs.	Depth, inches
2	1	2	2
3	$1\frac{1}{8}$	3	2
4	$1\frac{1}{8}$	4.5	2
5	$1\frac{1}{8}$	5.5	$2\frac{1}{2}$
6	$1\frac{1}{8}$	6.5	$2\frac{5}{8}$
7	$1\frac{1}{8}$	8.5	$2\frac{5}{8}$
8	$1\frac{1}{8}$	9	$2\frac{3}{4}$
9	$1\frac{1}{8}$	11	$2\frac{5}{8}$
10	$1\frac{1}{8}$	12.5	$2\frac{5}{8}$
12	$1\frac{1}{8}$	14	$2\frac{5}{8}$
14	$1\frac{1}{4}$	16	3
15	$1\frac{1}{4}$	18	3
16	$1\frac{1}{4}$	20	3
18	$1\frac{3}{8}$	22	3
20	$1\frac{3}{8}$	25	$3\frac{3}{8}$
24	$1\frac{3}{8}$	36	$3\frac{3}{8}$
30	$1\frac{1}{2}$	45	$3\frac{3}{8}$
36	$1\frac{5}{8}$	60	$3\frac{5}{8}$
42	$1\frac{5}{8}$	75	$3\frac{3}{4}$

The caulking of lead wool must be done by experienced men or by a pneumatic caulking machine. A heavy hammer weighing not less than 4 lbs. should be used, and the joint should be finished so as to be $\frac{1}{8}$ in.

inside of the bell. When the joints have been made, the pipe-line should be tested by air under a pressure of not less than 800 lbs. per sq. in.

The advantages claimed for lead wool joints are: a saving in material, the dispensing with the melting pot, and great density of the joint. For pipes of less than 24 ins. in diameter, lead wool joints cost more than cast lead joints, but for larger pipes the cost of both kinds of joints is about the same.

The quantities of lead wool and yarn required in joints of cast iron pipes are approximately as given in Table XXVII.

65. *Leadite** is a composition of iron sulphur, slag and salt, which is finely ground, melted and poured as a substitute for lead in joints of water and gas mains. The weight of leadite is only 118 lbs. per cubic foot, when melted. Lead weighs about 708 lbs. per cu. ft. One ton of leadite will make about 5 times as many joints as a ton of lead.

Leadite melts at a temperature of about 400° F. It is melted in a suitable gasoline or kerosene furnace, and is poured like molten lead. As it expands in cooling and forms a vitreous, water-tight substance, it requires no caulking. Leadite is shipped in powder form in sacks and barrels, containing, respectively, 100 and 350 lbs. of the material. When the joints are properly made, the leadite can be used for pressures up to 250 lbs. per sq. in. Leadite is more elastic than lead and, consequently, is not squeezed out of a joint like lead, when a pipe-line settles or is jarred.

The following table, furnished by the Leadite Company, gives the quantity of lead and leadite required to make joints, 2½ ins. deep, in ordinary water pipe.

TABLE XXVIII. — QUANTITIES OF LEAD AND LEADITE IN PIPE-JOINTS

Size of pipe, inches	Lead, pounds	Leadite, pounds	Size of pipe, inches	Lead, pounds	Leadite, pounds
4	6	1½	20	35	8½
5	7½	1¾	24	45	11
6	10	2½	30	60	15
8	12	3	36	80	20
10	15	3¾	40	90	23
12	18	4½	42	100	25
14	22	5½	48	120	30
16	25	6	60	200	50
18	30	7½	72	260	65

NOTE. — For high pressures the quantities of leadite given in the above table should be slightly increased.

* Manufactured by the Leadite Co., Inc., Philadelphia, Pa.

Some of the water works in which leadite has been used are given in the following table:

TABLE XXIX. — SOME WATER WORKS IN WHICH LEADITE IS USED

Location	Diameters	Lengths in miles	Pressures in pounds per square inch
Atlantic City, N. J.	4–20	8	....
Wilkinsburg, Pa. (Pennsylvania Water Co.)	4–30	6	100–175
Jacksonville, Fla.	6–10	6	62–110
Akron, Ohio	4–30	....	90–127
Sunbury, Pa.	4–24	....	60– 90
Hopkinsville, Ky	4	$\frac{1}{4}$	

66. Pipe-laying. — The trenches for water pipes must be dug to such a depth as to provide a sufficient covering of refilling to protect the pipes from injury from passing loads and against freezing. In the latitude of New York a covering of 4 ft. of earth is sufficient, but, in cold countries, the pipes have to be laid about 7–10 ft. below the surface, to protect them against freezing. The width of the trench should be sufficient to enable the caulkers to perform their work with care. For this reason the minimum width of a trench for small pipes in good ground should not be less than $2\frac{1}{2}$ ft., and for larger pipes a space, one foot wide, should be provided on either side of the pipe, the ordinary width of a pipe trench being, therefore, practically equal to the inner diameter of the pipe plus 2 ft., with a minimum width of $2\frac{1}{2}$ ft.

Pipes up to about 12 ins. in diameter are placed on the bottom of the trench, the extra space required for the bells being excavated at the required intervals. Larger pipes are usually laid on wood blocking, and are held in place by wooden wedges. Two blocks are provided for each pipe, one being placed near the bell and the other near the spigot. For a 48-in. main, the blocks are usually 6 ins. × 12 ins. × 4 ft. and the wedges are 6 ins. wide by 4 ins. deep at one end, one-half inch thick at the other end, and 18 ins. long. The blocks give the pipes a good bearing on the bottom of the trench.

Curves of large radius can be made with ordinary straight pipe — short lengths being used, if necessary — by laying the pipes as chords of an arc. In doing so, care must, however, be taken to allow sufficient space at all points of the joint for pouring and caulking. For sharper curves special curved pipes are used.

67. Universal Pipe (Fig. 38). — A new type of bolted joints for cast iron pipes was introduced about 1904 by the Central Foundry Company of New York. The pipe having these joints is known as *Universal Pipe,* and is manufactured in sizes of 2 to 16 ins. in diameter. The joints are turned in a lathe, by means of specially designed tools, to a taper, but that of the spigot ends is made slightly less than that of the bell ends. By this arrangement a perfectly tight joint is obtained for water pipes under pressure as high as 300 lbs. per sq. in. by simply drawing the spigot ends into the bell ends, no gaskets, lead, cement, oakum or any other filler being required. The differential taper gives considerable flexibility to the joints, without causing leakage, and takes care, also, of expansion and contraction.

Universal pipe is made of a good grade of cast iron. For adaptability to all general purposes it is made in standard lengths of 6 ft. Curved lines are easily laid, and the use of specials can be avoided in many cases. For low pressures the pipes have two wrought iron

FIG. 38. — Universal Pipe.

bolts at each joint, but for high pressures four bolts are provided at each joint for pipes of 12 ins. or more in diameter. Universal pipe can be laid by two men, each provided with a wrench, preferably of the ratchet type. White lead mixed in the oil is used at the joints as a lubricant.

Universal pipe can be laid without difficulty in wet trenches and, once drawn tight, remains tight. Branch pipes can be easily connected to the main pipe. This type of pipe has been used successfully in a number of cities. In the high pressure fire system of Philadelphia, more than 30 miles of this kind of pipe have been laid.

68. Manufacture of Cast Iron Pipes. — In the following pages we shall describe briefly the manufacture and testing of cast iron pipes, discussing the different steps in the order in which they occur at the foundry.*

* "The Manufacture and Inspection of Cast Iron Pipes," by Thomas H. Wiggin, Jour. Assoc. Engineering Societies, 1899, XXII, p. 209.

69. *Molding.* — There are three general methods of molding castings, known as the *green sand, dry sand,* and *loam method.* In the first, which is used for small castings, or large ones of simple shape, slightly moistened sand is rammed around a pattern in a wooden or iron box, called a flask. The surface of the mold is usually smoothed over with a mineral facing (powdered graphite, etc.), which prevents the iron from penetrating the sand, insuring thus a smooth surface for the casting. If a core is required for the casting, it is made by ramming green sand into a *core-box,* and coating it afterwards with graphite, etc. As such a core possesses, however, but little strength, a *baked core,* made as described hereafter, is generally used for the green sand method.

The dry sand method is used for molding standard length pipes and moderately large and complicated castings. It insures a smoother surface and a truer shape than the green sand method. The mold is made in a flask, as in the first method, but the sand placed next to the pattern has its cohesive strength increased by the addition of flour and clay water. The surface of the mold is coated with a liquid blacking, containing carbonaceous material (such as powdered coal and graphite, mixed with clay water or molasses, etc.), and the whole mold is baked in an oven at a moderate temperature for about twenty-four hours.

The core for the straight pipes is formed by winding a twisted hay rope* around a pipe or iron spindle having many holes to allow the gases to escape during the casting. The sand, prepared in the same manner as for the mold, is plastered against the hay, brought to the proper shape and coated. After being properly coated, the core is baked in an oven.

The loam method is used for large, heavy castings and for such which are formed without a pattern by revolving a form (*sweep*), which cuts out the proper shape in the sand. In this method, a mold is built around the pattern with soft bricks, iron binding plates and mud made of clay, sand, wheat flour, clay water, etc. A skim coat of fine mud is put next to the pattern, the surface is coated, and the whole mold is baked as in the dry sand method. Sometimes a cast iron flask is used, in which a lining of split bricks and loam are placed. The core is prepared as in the second method.

The choice of the method of molding, to be adopted for any given case, will depend on the number of pieces of the same pattern to be cast, the available fittings in the foundry, the smoothness desired in the casting, and somewhat on the judgment of the founder. Caps, covers, etc., which require no core, are made by the green sand method.

* A few manufacturers use excelsior instead of salt hay, where the latter is too expensive or difficult to obtain.

Sleeves are, also, usually made by this method, with green sand cores. All straight pieces are made in dry sand with baked cores. This method is, also, generally used for reducers, blowoffs, tees, etc., except for large sizes, when they are cast in loam.

Many cylindrical castings can be made without a pattern by means of a *sweep*, a board revolving on a vertical axis, and having a cutting edge which is given the proper form for cutting out in the sand the shape of the desired castings. Loam is always used for this method. Curved pipes of large diameter are also frequently made by means of a sweep. The spindle on which the sweep revolves is bent to the shape of the center line of the pipe. A small sweep is used, and the position of the collar which attaches it to the spindle is changed about every six inches. This method will, of course, not produce a perfect curve, which would require a continuous motion of the sweep along its spindle. Any apparent angles that appear are smoothed over by the molder by hand.

70. *Molding Straight Pipes.* — Formerly straight pipes were generally cast on their sides, the result being often an uneven thickness of metal. According to the present practice, pipes are always cast with their axis vertical, and, except for small pipes, with their bell ends down. In Fig. 39 we show the mold of a straight pipe. A cast iron ring *B*, called the chill, forms the foundation for the whole mold. Its name is really a misnomer, as it is covered with molding sand, to prevent the melted iron from coming in contact with *B*, and being chilled. On the chill is placed the flask *A* which holds that part of the mold which forms the outer surface of the casting. This flask is placed in a vertical position, and a lining of sand *J* is rammed between the flask and the pattern. The pattern is then removed, and the sand is coated with a suitable metallic blacking. The bead-ring *F*, at the top of the mold, is made by pouring semi-fluid mud (made extra strong by the addition of clay water and wheat flour) into an iron mold, in which it is baked. To obtain sufficient strength, an iron ring is usually incorporated into the bead-ring. Six to a dozen semicircular nicks are cut in the edge of the bead-ring, next to the core, to permit the iron to flow into the mold. There is, also, a single nick (called the *riser*) in this edge, which allows the air and gases to escape as they are displaced by the entering iron. The melted iron is poured into a trough of sand, called the *runner*, which rests on the top of the flask.

The core is made by revolving by power an iron spindle *C*, which is to form the center of the core. A twisted hay-rope *E* is first wound around the spindle. Then mud consisting of argillaceous loam, sand, and water, which are mixed in a pug mill, is plastered over the hay-rope. As the spindle revolves a long wooden straight-edge (called a *strike*), bound with iron, is brought to bear against the mud and cuts the core, as in a lathe, to the desired diameter.

The first coat of mud brings the core within about half an inch of its final diameter. It is baked over night, and then receives a skim coat of a fine, more sandy material, and is finally coated with liquid blacking, which gives the core a very smooth surface. This blacking is usually applied by means of the *strike*.

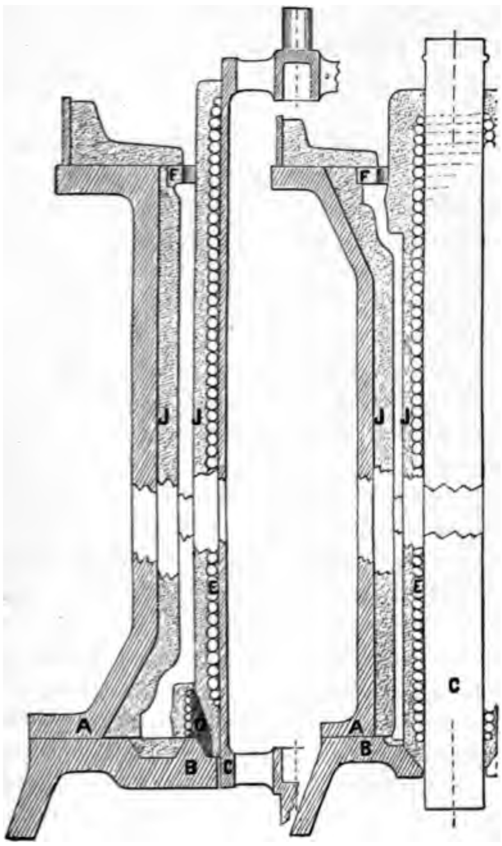


FIG. 39. — Mold for Cast Iron Pipe.

If it is desired to increase the thickening of the walls of a pipe, for which the patterns have already been made, it can be accomplished in two ways: by reducing the diameter of the core; or by making a new pattern for the outside of the pipe. The former plan involves practical difficulties in the manufacture of the pipe, and has the undesirable feature of giving the pipe a diameter including a fraction of an inch. The latter way is, therefore, preferable.

71. *Casting.* — The iron is melted in an ordinary cupola, coke being generally used as fuel, and oyster shells or limestone being added to act as flux. The proportion of fuel to iron varies, being on an average about one to eight by weight. A new fire is started each day. After the casting has been done, the bottom of the cupola is dropped. The remnants are run through a revolving cinder-mill, which separates the iron from the cinder. The iron which is thus obtained, known as *shot-iron,* is mixed sparingly with new charges of iron, as it contains, like cinder-iron, an extra proportion of impurities. The risers, runners, etc., and rejected castings are broken up and charged in like manner into the cupola.

The melted iron is tapped from the cupola into a pouring ladle, a large bucket holding three to six tons. If the founder considers the iron too hot, that is, hot enough to injure the mold, it is either allowed to cool a little, or else some scrap iron or pig is added to reduce the temperature. The ladle is then skimmed of dross, and a little earth is thrown in at the nose of the ladle to act as a dam to keep back the newly formed dross. The iron is poured from the ladle into the *runner,* which should be kept full, in order that the dross that floats on the surface of the iron may be kept back by the gates. In the ordinary method of casting, the iron flows from the runner into the top of the mold at the spigot ring. This obliges it to fall about a height of twelve feet at the start. To avoid the shock thus produced, the iron is sometimes led through a sand-lined passage to the bottom of the pipe, until it has risen above the hub of the pipe, when it is allowed to enter from the top as in the first method. The main objection to the latter method is, that a cool scum is formed on top of the iron entering at the bottom and prevents the iron, coming afterwards from the top, from uniting with the iron at the hub. Both methods of casting are used successfully.

It is very important to provide sufficient vent for the steam and gases that arise when the red-hot iron comes in contact with the mold, and to permit the air to escape from the mold. This object is partially accomplished by the risers at the top, but, in addition to this, holes must be placed in the flask and in the spindle of the core. During the pouring of the metal, the gases burn in blue flames at these vent holes. In order not to retard the escape of gases, the founder has to be careful not to use molding earth of too compact a texture.

72. *Turning out and Cleaning the Pipes.* — After a pipe has been cast, it is left to cool for a sufficient time, and the spindle is then pulled out. If it were not for the fact that it had been surrounded by a hay rope, this could not be done as the iron, in shrinking, would hold it firmly. The elasticity of the hay rope, while permitting the iron to shrink, prevents it from holding the spindle. After the spindle has been removed, the pipe is left for some time in the flask, usually over night. Pipes of less than 16 ins. diameter are sometimes turned out while still red, but this is apt to crack them by the strains caused by sudden cooling.

The pipes come from the casting pit with the iron socket ring *D*, in the bell, projecting gates and fins, and all of the core-earth inside of the barrel. The pipe must be thoroughly cleaned and the fins, gates, etc., must be chipped off. Four men will ordinarily clean about fifteen 48-in. pipes per day.

73. *Inspection.* — After the pipes have been cleaned, they are inspected. There are a great number of causes of imperfections which necessitate the rejection of a pipe. The imperfections ordinarily found in pipes may be classified as follows:

1. Mistakes in dimensions, due to incorrect patterns, wrong setting of strikes for cores, etc.

2. Too small diameter, due to abnormally high shrinkage. This occurs rarely, and only with large pipes.

3. Scabs, which are excresences on a pipe, caused by a piece of the mold coming out. If the earth floated to the top of the pipe, there would be no trouble on account of the scab, but there is always a danger that the earth has remained somewhere in the pipe, causing a weak spot. If the scab is of considerable size, it is advisable to reject the pipe.

4. Mold-cut. This is caused by the iron penetrating the mold. The dirt does not float away, but remains partly in sight. If the mold-cut is of considerable size, the pipe should be rejected.

5. Socket-cut. This is a mold-cut in the socket, and is usually fatal.

6. Dirt falling into the *runner* when it is made. There is usually a little of this, but it leaves no tell-tale.

7. Core-swells and core-strains, causing imperfections on the inside of the pipe. They come from insufficient strength in the cores. If the core should yield slowly, after a crust had been formed all around the pipe, and the gates had become too hard to admit any more iron, a settling would occur in the inside of the pipe, causing a cavity.

8. Scale on the inside of the pipe. It forms sometimes in blotches, about $\frac{1}{16}$ in. thick, and leaves a shoulder when it comes off. The causes of this defect are not well understood.

9. Imperfect spigots are caused by the dross and dirt in the iron which floats on the top. The remedy is to cut off the spigot end and to shrink a wrought iron bead around the pipe into a groove cut around the pipe. Modern practice is to provide a shrink-head, and to cut off to a correct length.

10. Shrinkage cracks. They are comparatively rare and usually short.

11. Excess or lack of weight. A pipe should always be rejected if its weight is below the minimum called for, but there is no objection to taking a pipe weighing somewhat too much, if the founder does not charge for the extra weight.

12. *Out of round.* Occasionally a flask will distort the pipe, probably changing its shape after the spindle has been withdrawn.

13. Uneven thickness, caused by misplacing the core. It may be detected by caliper measurements, or by watching the pipe rolling on smooth, level skids. The heavy side will go down rapidly and tend to stay down. This defect occurs rarely in pipes cast vertically.

14. Unsuitable iron. The iron should be soft enough to cut or drill, and not cold- or red-short (too hard or too brittle).

15. Leaking or bursting when tested in the press.

16. Cracked in handling.

In examining pipe, the inspector tests the hub and spigot ends by means of templates and the thickness of the pipe by calipers. He pecks strongly with a pointed hammer all around the spigot end, and at all doubtful points. The inspector also examines the pipes for hardness, in order to judge whether or not it can be drilled or cut.

74. *Coating.* — After the inspector has accepted the pipes, they are rolled to the place where they are coated. As already stated, pipes are almost universally coated with Dr. Angus Smith's coal-tar varnish. There are some minor differences in the materials used in the composition, and in the methods of applying them. The pipes are brought, in an oven, to a temperature of 300 to 350° F., and are then immersed in a cauldron containing the varnish which is kept at a heat of 200 to 225° F. After being left in the hot bath for a certain time, the pipes are lifted vertically out of the liquid, the surplus of which runs off the pipes.

In the large foundries near Philadelphia the pipes are coated as follows:

The pipe is heated in an oven so constructed that the heat is uniformly distributed. Electric pyrometers regulate the temperatures of the oven and of the tar bath. The pipe is lowered into the tar bath, and left there from three to ten minutes, according to the condition of the work about the vat. The composition used consists simply of crude gas-tar, to which some dead oil of tar is sometimes added.* The pipe is then taken out of the bath, and is allowed to drain over the vat, the draining being assisted by scraping the invert with a segmental hoe. The pipe is then put on skids and imperfections in the coating are remedied by smoothing the coating or adding some of the

* Dead oil is that part of coal-tar which is obtained in a fractional distillation between about 410–750° F. It contains both the creosote and the anthracene oils.

tar varnish by means of a brush or mop, as may be required. According to the conditions, it requires from one-half to two hours for the coating to become hard.

The coal-tar varnish applied to water mains does not always protect them from rusting. Many engineers think that the varnish used nowadays is inferior to that applied thirty or forty years ago, and attribute this difference to changes which have taken place in the making of gas. Some manufacturers of pipe buy the coal-tar they use for coating pipes in the market, wherever they can get it the cheapest, without having any chemical analysis made. Tar from different works vary, however, in physical and chemical character. The mixture of coal used in making the gas, the heat in the retorts and condensers, etc., have an effect on the tar, which is a complicated mixture containing more than one hundred different chemical compounds.

It is, therefore, very evident that the quality of the coal-tar varnish put on water mains differs very much, according to the composition of

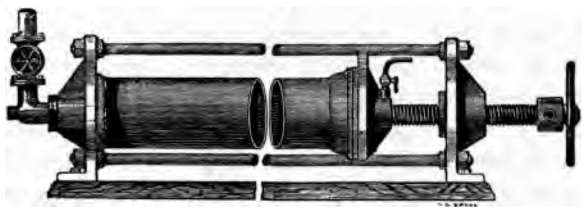


Fig. 40. — Hydraulic Press for Testing Water Mains.

the tar used. The temperature of the pipe, when taken out of the bath, has, also, a great deal to do with the quality of the coating. If the pipe is too hot, the coating will be over distilled, and will, probably, be too brittle. If the pipe is too cool, the coating will be too thick and will not harden sufficiently. Thin pipes should be heated hotter than thick pipes, as they will not retain the heat as long as the latter. The distillation must, therefore, be more rapid at first.

75. *Weighing.* — After the pipes have been coated, they are weighed on a suitable platform-scales, in the presence of the inspector, the weight, class and number of the pipe being painted on the inside with white lead.

76. *Proving.* — The pipes are next subjected to hydraulic pressure. Each pipe is placed between two heavy discs (Fig. 40), one being stationary and the other attached to the piston of a hydraulic cylinder. Gaskets are placed between the ends of the pipe and the discs, to insure water-tightness. Water at a low pressure is admitted into the pipe through an orifice in one of the discs, the air in the pipe escaping through another orifice at the top of the disc. It is important to expel practically all of the air in the pipe, as it might cause accidents, on account of its expansive force, by throwing fragments of a pipe, should it burst in the press. When there is only water in the pipe, the bursting can cause no danger. After the pipe has been filled with water, the air-valve is closed and a high pressure, amounting to 300 lbs. per sq. in., is turned on. While the pipe is subjected to the pressure, the inspector hits the pipe several blows at different places, to see whether any cracks are started. But few pipes burst in the press.

After a pipe has passed the pressure test successfully, it is ready for shipment, and the inspector allows his initials to be painted inside the pipe.

Good foundries are arranged on a gentle slope, in order to facilitate moving the pipes, one process following the other in regular order.

77. *Tests for the Iron.* — Three qualities of cast iron must be considered: strength, hardness, and durability. The degree to which a casting should possess these qualities depends upon the use to which it is to be put. The effect of the chemical composition of the iron on the qualities mentioned above is discussed on pages 94 to 97.

Pipe castings should be strong enough to bear shocks caused in handling the pipes, from water-hammer, etc.; they should be capable of being readily cut and drilled, and as durable as possible, without interfering with the requirements first mentioned. Hard iron is preferred for pipes by some engineers, as it resists the chemical action of earth and water better than softer iron, but hardness is a disadvantage if the pipe is to be cut or tapped. The fitness of the iron for being cut, drilled, etc., can be tested directly with a chisel, file, or pointed hammer. The general quality of the iron can be judged from the appearance of its texture, when broken. Thus, a white texture indicates hardness and brittleness; a fine gray texture is a sign of strength, while a coarse, crystalline fracture with much graphitic carbon indicates weakness. In judging of the quality of the iron, the indications of the fracture must be taken in connection with chemical analysis, as the rate of cooling has much to do with the appearance of the fracture, while most of the alloys may remain unchanged.

In testing a bar of iron from the same iron used for the pipes, it is not sufficient to determine only its strength to resist tension or compression, as very brittle iron may possess such strength to a high degree. It is essential to note the elasticity of the iron (shown by its strength under tension), as this quality enables it to resist shock. The best way of testing a bar of iron is under a transverse load, as its deflection indicates its elasticity. It is, also, the cheapest way, as it does not require any complicated apparatus. This method was adopted for the Metropolitan

Water Works of Boston. Specimen bars, 1 × 2 × 26 ins., laid flatwise with a clear span of 24 ins., were required to carry a center load of 1900 lbs., with a deflection of at least 0.3 in. This test was required as a daily average from a cupola.

In taking a test-bar, it must be borne in mind that the strength of this bar may differ materially from that of the castings. The strength of cast iron depends largely upon the rate at which it cools, quick cooling producing strength and brittleness, while slow cooling produces softness and elasticity. The surface of the iron, called *the skin*, has

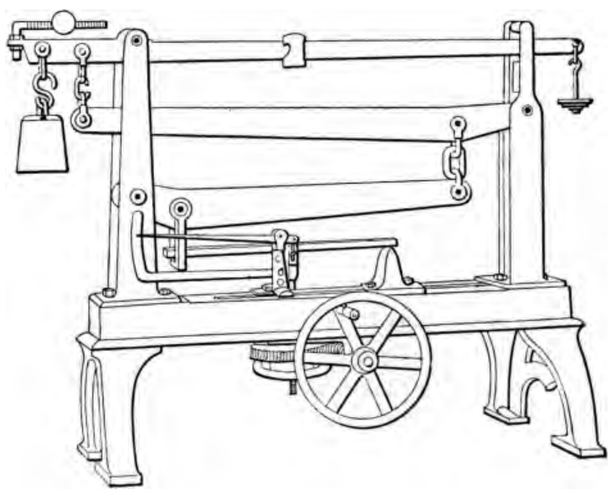


FIG. 41. Riehlé's Testing Machine.

greater strength than the inner portions of a bar. For this reason, if a test-bar is planed down to the size of the smaller bar, the latter will be found to be the stronger of the two, when tested; while the former is more elastic, on account of the removal of the hard skin. It follows that the test-bar should equal in thickness the thickest part of the proposed casting.

Although the objections noted above hold against using small sample castings, as a means of judging the strength of a larger casting, the method of test-bars is generally used, for lack of any better method. The founders are usually permitted to cast the test-bars in any way they prefer (in green sand, dry sand, etc.), the only requirements being that the iron must be the same kind as that used for the pipes, and that the test-bars shall not be annealed.

The bars can be tested in a machine made by Riehlé Bros. of Philadelphia, Pa. (Fig. 41), the load being gradually applied at the center of the bar, and the deflection being noted. The errors that may occur in the testing may be neglected in view of the roughness of the whole test-bar method.

As pipes, about one-half to one in. thick, appear to have about the same texture as a 1 by 2-in. bar, these test-bars indicate, very probably, the strength of the iron in the pipes.

Although the transverse test may not give the exact strength of the iron in the pipe, it serves to detect variations in the iron in the cupola, due to differences in charging. Taken together with a careful examination of the texture of the fractured sections of test-bars, it forms a good indication of the quality of the iron. Two bars should always be cast from the same ladleful of iron, and tested, in order that the exactness of the results obtained may be checked.

78. *Chemical Tests.** — As the chemical composition of a piece of cast iron determines largely its character, it would seem that the composition of the iron should be specified, and that a chemical analysis should be included in the prescribed tests. Thus far, this has not been generally done, as there is still some difference of opinion among foundry-men and chemists about the èxact effect which results from certain combinations of the different substances usually found in iron. A uniform chemical analysis should, however, not be insisted upon for all places of manufacture, as the variation in the chemical composition of the ores from which the pig iron is made in different places must be considered.

The general effect which each of the foreign substances contained in iron ore produce is well understood.

79. Foreign Elements in Iron. — The elements usually found in pig iron are: iron, carbon, silicon, sulphur, phosphorus, and manganese, the two first mentioned having the controlling effect.

80. *Carbon.* — This element is found in cast iron, either combined or uncombined, as graphite. Combined carbon is usually admitted to embrace more than one kind of carbon. A great deal remains still to be discovered about the effect of carbon on iron. Its action depends not only upon the amount present in the iron, but, also, upon its condition. Mixtures in which charcoal iron is the chief component show perceptible differences from those made entirely from coke-iron. Graphite is known to be the principal softening agent in cast iron.

81. *Silicon.* — Thirty or forty years ago, foundry-men regarded this element as their worst enemy. It has since been shown that silicon has the very beneficial function of promoting the formation of graphite,

* "Modern Cupola Practice and the Physics of Cast Iron," by Bertrand S. Summers, *Eng. News*, Oct. 6, 1898, p. 219.

while lowering, at the same time, the saturation point of iron for combined carbon. Some foundry-men claim that an increase in silicon causes always a proportional increase in graphite, and that silicon may be blindly added to a foundry-mixture, without considering other conditions. Analyses of castings, given by Mr. Bertrand S. Summers of Chicago, Ill., prove that this view is erroneous, and that there are other conditions that affect the amount of graphitic carbon as much, and possibly more, than the silicon. It is, therefore, more important to control the amount of graphite present in the pig iron put in the cupola than the amount of silicon which it contains.

It is now generally accepted that the silicon affords the pipes considerable resistance to corrosion, and, for this reason, it is claimed that cast iron pipe resists corrosion better than steel pipe. Some silicon should always be present in a casting. For most foundry purposes the iron should contain at least $1\frac{1}{2}$ per cent of silicon. A machinery-casting of close structure, that is to be soft enough for tooling, should not contain over $1\frac{1}{2}$ to $2\frac{1}{4}$ per cent of silicon.

For light hardware, which does not require to have great strength, the silicon may amount to 3 per cent. The effect of this large amount of silicon is to make the iron very fluid, so that it will take delicate molds well, and avoid the difficulty due to shrinkage. If more than 3 per cent of silicon is carried, the casting becomes too brittle.

82. *Sulphur* frequently counteracts the good effects of silicon, and causes much trouble in the foundry. The brands of soft charcoal or coke-iron do not generally contain enough sulphur to cause any trouble. The excess amount of this element is in many instances introduced into the mixture through the ferro-silicon irons which are added to the charge. In good foundry-practice the amount of sulphur in a casting is limited to 0.08–0.10 per cent.

83. *Phosphorus.* — For the greater part of foundry work, the presence of this element, up to 1 per cent, is an advantage, as it keeps the metal fluid. Phosphorus makes the iron, however, brittle and very liable to break under a sudden blow. Where great strength and resilience are required, the amount of phosphorus should be kept at less than 0.5 per cent.

84. *Manganese* is an important element in iron. Metallurgists consider it to have a beneficial effect, as it promotes combined carbon. If silicon predominates in the iron, it would tend to counteract this effect. Manganese increases the strength of the iron, closes the grain, and reduces the tendency to form blow-holes.

85. *Oxidized material,* especially rusty scrap, has considerable influence upon the mixture in the cupola. The surface rust in pig iron is usually not noticeable, but if the corrosion has gone to considerable depth, it has a very bad effect in the cupola. When such light scrap, having a large surface exposed to atmospheric corrosion in proportion to its volume, is charged in the cupola, the casting made will consist of exceedingly dirty and spongy iron. The cleaner the materials put into the cupola are, the cleaner will be the castings.

86. *Effects of the Blast.* — The blast has, in all probability, a great influence in furthering the mutual reactions of the different elements in the cupola. Although the exact effects of this influence has not yet been clearly shown, increase of blast seems to have a decided tendency to increase the total amount of carbon in the cast iron, augmenting correspondingly the percentage of graphite.

87. *Iron Specifications.* — As already stated, it has not yet become customary to specify the limits within which certain elements, usually found in iron, are to be kept, although it is sometimes done.

The best composition of iron for cast iron pipes depends upon the size of the pipe and upon the thickness of the metal. R. D. Wood & Co., of Philadelphia, Pa., give the following limitations in percentage for foreign elements in the average quality of cast iron used for pipes: silicon, 1.00 to 2.50; sulphur, 0.08 to 0.15; phosphorus, 0.50 to 1.00; manganese, 0.40 to 0.80.

CHAPTER VI

WROUGHT IRON AND STEEL PIPES

88. Historical Notes. — Prior to 1853, wrought iron was rarely used for water mains in the United States, as cast iron pipes were cheaper. Wrought iron penstocks and supply pipes for turbines were, however, occasionally constructed, and some of these pipes were in use for more than fifty years.

Wrought iron pipes were first used on a large scale in 1856 in the hydraulic mining operations in California, owing to the large expense involved in transporting cast iron pipes in mountainous regions. For about three years prior to this time, thin sheet iron pipes, 5 and 6 ins. in diameter, had been laid to lead water from flumes or ditches to the large jet nozzles used in hydraulic mining.

The first wrought iron pipes laid in California were made of very thin plates, the metal being occasionally strained to half its breaking strength. As the plates rarely exceeded No. 10 Birmingham Wire Gauge (0.134 in.), the small rivets used for these pipes could be driven cold. Except for great pressure, the seams were all single riveted. The pipes were often made right at the mines in sections, 18 to 25 ft. long, with slightly tapering courses. They were dipped into a bath of melted asphalt and coal tar, to protect them against corrosion, and were shoved together, stove-pipe fashion, by means of jack-screws, the asphalt coating at the joints being first softened by heating. These joints were not secured by any riveting. If leakage occurred, it was stopped, or at least reduced, by throwing sawdust into the pipe.

One of the oldest of these conduits was a line of 40-in. wrought iron pipe, laid in 1857 across a depression at Timbucto, Yuba County, Cal. During the following decade various other lines of similar pipe of a temporary character were laid, and the use of wrought iron conduits was, also, extended to water works, thicker plates being used and the rivets being driven hot. For small pipes the joints were made by means of sleeves and hot lead. In larger pipes the circumferential joints were riveted, no provision being made for expansion and contraction, as it was deemed unnecessary.

In the Western States, the largest amount of wrought iron pipe was laid to convey water to San Francisco. The first pipes for this purpose were constructed about 1862. By the end of 1892, about 72 miles of wrought iron pipe, 22–44 ins. in diameter, had been laid, in connection with the water works of San Francisco, besides a submarine line of 16-in. pipe, 13,800 ft. long, across San Francisco Bay, in 55 ft. of salt water. Some of the early lines of wrought iron pipe are mentioned in the following table:

TABLE XXX. — RIVETED WROUGHT IRON PIPES

Year	Locality	Diameter, inches	Length, feet	Thickness of metal, inches	Maximum head, feet
1868	Humbug Canon, Cal.	26	1,200	$\frac{1}{16}$	120
1869	Cherokee Gravel Mines, Cal.	30	13,200	$\frac{3}{8}$	850
1870	Spring Valley Mining Co., Cal.	30	14,000	....	....
1872	Virginia City Water Co., Nev.*	11½	37,100	$\frac{5}{16}$	1720
1888	Tuscarora, Nev.	8	68,640	....	900

* This pipe leaked considerably at the circular joints. The following year a 10-inch lap-welded pipe with screw couplings was laid alongside of the riveted pipe. Although the second pipe had a smaller diameter than the first one, it delivered as much water, owing to its smooth inner surface.

The wrought iron water pipes of San Francisco, and several of those mentioned in the above table, were designed by Herman Schussler, M. Am. Soc. C. E., the Chief Engineer of the Spring Valley Water Co.

89. *In the Eastern States*, the first riveted wrought iron pipe of importance was laid for the Rochester Water Works in 1873 to 1875. This pipe-line is 36 ins. in diameter and 9¾ miles long. According to the original intention, cast iron pipes were to be laid in this line, but the prices bid were found to be prohibitory, and it was, therefore, decided to make the pipes of wrought iron. Although this pipe-line has been more or less corroded in places, it is still in service.

TABLE XXXI. — PIPE-LINES OF RIVETED STEEL

Year	Water works	Location	Diameter, inches	Length, miles
1892	East Jersey Water Co.*	New Jersey	48	21
1894	Rochester Water Works †	New York	38	26
1895	Cambridge ‡	Massachusetts	40	3
1896	New Bedford §	Massachusetts	48	8
1895	Allegheny ‖	Pennsylvania	60	10
1906	Brooklyn Water Works	New York	72	8

* Jour. New England Water Works Assoc., 1893, VIII, pp. 18–42; *Eng. Record*, Aug. 8, 1891, p. 156; *Eng. News*, Oct. 11, 1890; Aug. 1 *et seq.*, 1891; and June 15 *et seq.*, 1893.

† *Eng. Record*, April 13 *et seq.*, 1895.

‡ *Ibid.*, Jan. 5, 1895, p. 97.

§ *Ibid.*, March 28, 1896, p. 293.

‖ *Eng. News*, Oct. 10, 1895, p. 234.

About 1890 soft steel began to supersede wrought iron for pipes, boilers, etc., on account of its greater cheapness. Some engineers claim that steel, when buried in the ground, corrodes more rapidly than wrought iron; but, according to the best authorities, this difference, if it exists, is not large, and steel has been used in all recent lines of riveted pipes, the most important of which are given in Table XXXI.

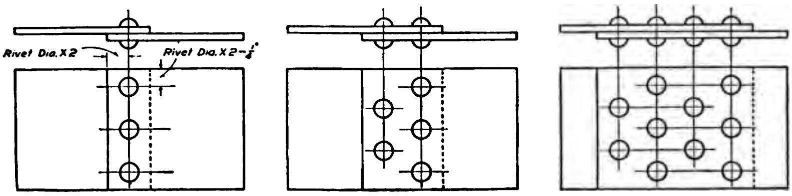
90. Large riveted steel pipes have been used extensively in recent years as conduits and penstocks for water power development, and as inverted siphons on aqueducts. A steel penstock, 18 ft. in diameter, was constructed for the Ontario Power Company of Niagara Falls, N. Y. It is subjected to a head of about 20 ft., which gives the water a velocity of 15 ft. per second.

On the line of the great aqueduct which brings a supply of water from the Catskill Mountains to the City of New York, a number of valleys are crossed by inverted siphons of riveted steel pipe, 9 or 11 ft. in diameter, and made of plates $\frac{7}{16}$ to $\frac{3}{4}$ in. thick (p. 306). Similar siphons of riveted steel pipes, 7.5 to 11.5 ft. in diameter, were constructed on the Los Angeles aqueduct (p. 253).

91. Design of Riveted Pipes. — The thickness of shell required for a riveted pipe to resist a given pressure is calculated by the formula for a thin ring, used for a cast iron pipe. In fixing, however, the value of permissible stress in the pipe, we must bear in mind that, even in well-designed pipes, the strength at a longitudinal joint is only 54 to 75 per cent of that of the metal in the pipe-ring itself, according to whether single, double or triple riveting is used. In a pipe made of soft steel, having a tensile strength of about 60,000 lbs. per square inch and an elastic limit of 30,000 lbs. per square inch, the metal in the pipe may be safely stressed to 15,000 lbs. per square inch, but the longitudinal joints only to 8000 to 11,000 lbs., according to the number of rows of rivets, and the latter values must be used in the formula for determining the thickness of the metal of the pipe.

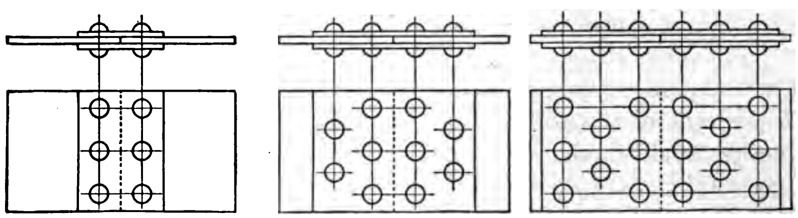
In designing a riveted joint, we have, however, not only to consider the question of strength, but, also, the practical requirement of making the joint water-tight. The questions how closely the rivets should be driven between centers for this purpose, and how near to the edge of the plate, are principally determined by experience. The formulas used for calculating the diameter and pitch (distance between centers of rivets) of the rivets for a pipe having a given thickness of metal are to a great extent empirical.

92. Riveted Joints. — The metal plates for a riveted pipe are joined either by lap-joints (Fig. 42) or butt-joints (Fig. 43), made with one or two covering-plates or straps. Occasionally a lap-joint is provided with a cover-plate (Fig. 44). For practical reasons lap-joints are only used for plates up to ½ in. in thickness. With thicker plates it would be impracticable to hammer the plates down, where necessary, at the joints, to obtain a close contact between the plates, and the rivets required, which are usually about twice the thickness of the plate, would be too large to be driven by hand. For a butt-joint twice as many rivets are used as for a lap-joint, and they may, therefore, have half the size. If only one covering-plate or strap is used, it should evidently have the same thickness as the shell of the pipe, but, when there are two, each need only have half the thickness of the shell. Butt-joints are somewhat more expensive than lap-joints.



Single Riveted *Double Riveted* *Quadruple Riveted*

FIG. 42. — Lap-joints.



Single Riveted *Double Riveted* *Triple Riveted*

FIG. 43. — Butt-joints.

Lap-joints may have one, two, three or more rows of rivets. With butt-joints at least two rows of rivets must be driven — one on each side of the joint — and as many more can be used as required. When the rivets are in line both longitudinally and transversely of the seam (Fig. 45), the joint is said to be chain-riveted. If they are ***staggered*** as shown in Fig. 46, the joint is said to be *zigzag* or ***staggered*** riveted.

93. Punching or drilling may be used for making the rivet holes, which are usually made $\frac{1}{16}$ in. larger than the diameter of the cold rivet, to allow for the expansion due to the heating of the rivet. As the rivet, if driven properly, fills the hole, its diameter is usually assumed in calculations of the strength of riveted joints to be equal to that of the hole. Some engineers assume, however, the diameter of the rivet to be that which it was when cold, as the rivets may be subjected to additional stresses caused by the rivet holes not being directly opposite to each other.

Punching diminishes the strength of the plates between the rivet holes, unless the plates are subsequently annealed, when the original strength is recovered. This may also be accomplished by punching the holes $\frac{1}{16}$ to $\frac{1}{8}$ in. too small, and reaming them to the required size. The fact that reaming restores the plates to their original strength shows that the injury done by punching extends only to a very short distance from the rivet holes.

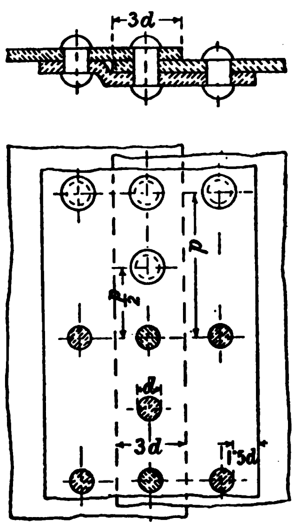


FIG. 44. — Combined Lap- and Butt-joint.

DOUBLE RIVETED LAP-JOINTS.

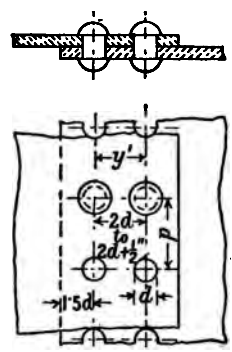


FIG. 45. — Chain Riveting.

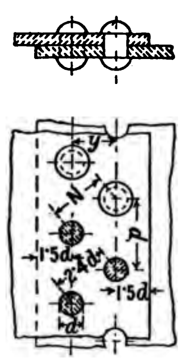


FIG. 46. — Zigzag Riveting.

Drilling is a slow and expensive process; annealing requires large furnaces and may injure the plate, if it is not done very carefully. Punching the holes too small and reaming them afterwards seems, therefore, the most practical method of securing a strong joint. In ordinary practice, plates having a thickness up to $\frac{1}{2}$ in. are punched. For greater thicknesses drilling, or punching and reaming, is resorted to. Experiments made by Prof. A. B. W. Kennedy * in England, and by others at the Arsenal at Watertown, Mass.,† have shown that drilling and reaming increases the strength of the metal on the line of perforations. This is explained by the condition of localized stress and consequent prevention of lateral contraction. The following table, compiled by Bindon B.

* Proc. Inst. M. E., 1888, p. 546.

† "Tests of Metals," 1886, pp. 1264, 1557.

Stony,* shows the effect on the metal between rivet holes of punching, annealing, and drilling.

TABLE XXXII. — RELATIVE STRENGTH OF PUNCHED AND DRILLED PLATES

Specimens	Unit strength of net section between holes, compared with that of the solid plate, taken as 100 per cent			
	$\frac{1}{4}$ inch	$\frac{1}{2}$ inch	$\frac{3}{4}$ inch	1 inch
	Per cent	Per cent	Per cent	Per cent
Punched	101.0	94.2	82.5	75.8
Punched and annealed	105.6	105.6	101.0	100.3
Drilled	113.8	111.1	106.4	106.1

For punched and reamed holes the same percentages may be used as for drilled holes.

94. Riveting may either be done by hand or by machinery, the power used in the latter case being steam, hydraulic pressure, or compressed air. Hand riveting, if properly done, will make as strong, but not as tight, a joint as machine riveting. The pressure under which the latter kind of work is done produces a greater frictional resistance between the plates, and requires, therefore, a greater force to produce visible slipping between the plates than hand riveting. When the rivets are driven by hand — as they have to be in many cases — they must be heated to the proper temperature, indicated by their being a bright red color, and the first few blows of the hammer must be strong and quick enough to make the rivet fill its hole entirely. In shops most of the riveting is done by machinery, as it is cheaper than hand work, and makes tighter joints. In machine riveting a pressure of about 35–50 tons is brought to bear on the rivet, and is kept on it until it is set. If a greater pressure is used, the rivet will be injured. Machine riveting is superior to hand riveting, if the rivet remains under a high pressure long enough to lose all the red color of heat. The force by which the plates are pressed together in machine riveting gives a joint considerable frictional resistance to sliding. This resistance depends, also, largely upon how the rivet holes were made. Where holes are punched to the full diameter, they are always slightly tapering, a noticeable burr being generally left on the lower side of the plate, and a corresponding depression on the upper side. Truly cylindrical holes can only be produced by reaming the punched holes, or by drilling.

With punched plates a greater frictional resistance will be produced, if the plates are assembled so as to bring the lower faces, containing the

* "Strength and Proportion of Riveted Joints," by Bindon B. Stony, London, 1885.

burrs, together, than if the smooth upper faces were placed in contact.* With the best work the friction between the plates may become equal to the shearing stress commonly adopted in designing iron and steel structures. On account of the numerous practical difficulties of accomplishing this with certainty, it is best to depend, for the strength of a riveted joint, entirely upon the rivets to resist tension, compression and shearing, and to consider the frictional resistance simply as an additional margin of safety.

Experiments have shown the ultimate compressive strength of wrought iron and soft steel rivets and plates to be 40–70 tons per square inch, although the same metal, when tested in direct, unconstrained compression, may have an ultimate crushing strength of only 25–30 tons per square inch. This is, doubtless, due to the constraint afforded by the overlapping rivet heads. In general, good wrought iron and soft steel rivets have a shearing strength of about 20 tons per square inch.

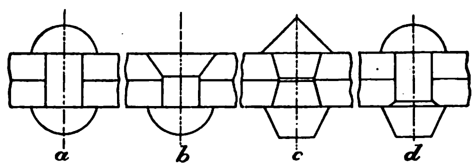


FIG. 47. — Rivet Heads. *a*, button-head; *b*, button- and counter-sunk heads; *c*, steeple- and pan-heads; *d*, button- and pan-heads.

The compressive strength of a riveted joint may have almost any value up to its empirical limit of two to three times its resistance to shearing.

In order to keep the diameters of the rivets within practical bounds, it is customary to assume the ratio of the compressive strength to the shearing strength of a riveted joint to be within the limits of 1 to 2. Practice in this respect varies greatly. Often the lower limit is chosen for a thick plate, to avoid getting the diameter of the rivets too large, while higher limits are taken for thin plate, to get this diameter large enough. Rivets greater than $1\frac{1}{8}$ ins. in diameter cannot be driven by hand, so as to make tight work. When the diameter is less than $\frac{1}{2}$ in., much skill is required to form even and concentric heads. Small rivets cool so quickly, that they are apt to be injured by being overheated in the forge.

When the rivets are in single shear, their diameter should be, in general, twice the thickness of the plate, while it should only be equal to the thickness of the plate, if the rivets are in double shear. In the latter

* This has been fully proved by experiments made by Bach in Germany, Considère in France, Prof. Kennedy in England, Prof. Talbor of Champaign, Ill., and by tests made at the Watertown Arsenal.

case, care must be taken to discriminate between the thickness of the plate and that of the straps.

Three different types of rivet heads (Fig. 47) are used. The first is called the pan-head or conical head; the second, called the steeple-head, is used for boilers and similar work; and the third type, known as the spherical, button-head, or snap-head, is used for riveted pipes and general structural work.

95. Caulking. — Riveted joints generally leak, unless the edges of the plates or cover straps are caulked. Formerly these edges were cut square and provided with a small groove, which was spread apart by means of a chisel. This kind of work, known as *split-caulking*, was apt to injure the plates, and a *round-nose tool* is now used for caulking. To facilitate the work, the edges of the plates or straps that are to be caulked

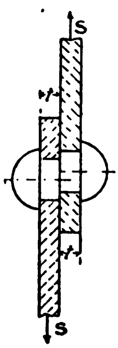


FIG. 48.

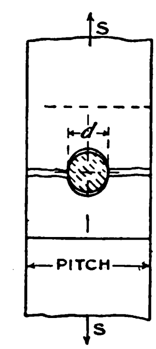


FIG. 49.

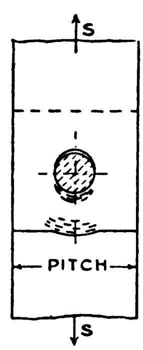


FIG. 50.

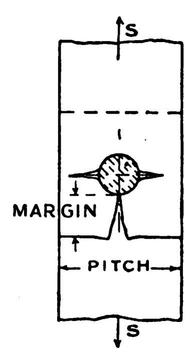


FIG. 51.

Failures of Riveted Joints.

are usually planed to a bevel of 1 in 3. Leakage occurs, sometimes, under the heads of the rivets, and must be stopped by caulking the edges of the rivet heads. Thin sheets of wrought iron or steel cannot be caulked, as the plates will bend or buckle under the blows of the hammer. While a good workman can caulk a sheet of metal as thin as $\frac{1}{8}$ in., $\frac{3}{16}$ in. is about the practical limit of thinness for sheets that are to be caulked.

96. Formulas for Riveted Joints.* — There are four ways in which a riveted joint can fail: (1) by the shearing of the rivets (Fig. 48); (2)

* "On Riveting, with Special Reference to Ship-work," by M. Le Baron Clauzel, Proc. Inst. M. E., 1881.

"General Formulas for Efficiency and Proportions for Riveted Joints," by Prof. J. H. Barr in Sibley Jour. of Engineering, October, 1900.

"Machine Design," by A. W. Smith and G. H. Marx, Fourth Edition, p. 113, New York, 1915.

"Machine Design, Construction and Drawing," by Henry J. Spooner, C. E., Third Edition, p. 143, London, 1913.

by the tearing of the plates between rivet-holes (Fig. 49); (3) by the crushing of the rivet, or the plate in front of it (Fig. 50), and (4) by the breaking of the plate in front of the rivet (Fig. 51). Theoretically a joint should be so designed as to have the same factor of safety to resist each of the four ways in which failure may occur. The diameter and pitch required in a well-designed joint can be computed by the following, simple formulas, in which all dimensions are to be taken in inches and all stresses in pounds per square inch. As the joint is composed of a number of equal units, each having a length equal to the pitch, *i.e.*, the distance from center to center of rivet, we shall only consider in the formulas one unit's length. Assuming all linear dimensions to be taken in inches, and all stresses to be given in pounds per square inch,

Let P = ultimate tensile strength of the unperforated plate;
T = ultimate tensile strength of the net section of the plate;
S = ultimate shearing strength of all rivets in the length of one unit;
C = resistance of metal, or metal around rivet-hole, to crushing;
d = diameter of rivet-hole, assumed as $\frac{1}{16}$ in. larger than the diameter of the cold rivet;
p = pitch of rivets;
t = thickness of plate;
f_t = tensile strength of plate;
f_c = crushing strength of rivets and plates, when the rivets are in single shear;
f_c' = crushing strength of rivets and plates, when the rivets are in double shear;
f_s = shearing strength of rivets, when they are in single shear;
f_s' = shearing strength of rivets, when they are in double shear.

97. *Lap-joints.* — The tensile strength of the unperforated plate is given by the formula:

$$P = ptf_t. \tag{47}$$

On the line of perforation the tensile strength is represented by:

$$T = (p - d)\,tf_t. \tag{48}$$

If the joint has only one row of rivets, the resistance to shearing will be:

$$S = \frac{\pi d^2}{4} f_s = 0.7854\, d^2 f_s. \tag{49}$$

The resistance the rivet, or the metal around the rivet-hole, offers to crushing will be given by the formula:

$$C = dtf_c. \tag{50}$$

To give the joint equal strength to resist every kind of failure we should have:

$$T = S = C.$$

If we equate the values of S and C, given by formulas (49) and (50), we find:

$$d = 1.27\, t \frac{f_c}{f_s}, \tag{51}$$

which gives the proper theoretical value of d for given values of t, f_c, and f_s.

By equating the values given for T and S in formulas (48) and (49) we obtain:

$$p = 0.7854 \frac{d^2 f_s}{t f_t} + d, \tag{52}$$

which is the proper pitch for given values of d, f_s, and f_t.

98. *For double-riveted lap-joints*, the unit strip of joint, whose length equals the pitch, will have two rivets, each in single shear. This will cause a modification of some of the formulas given for single-riveted lap-joints, and we shall have, when the joints are double-riveted:

Double-Riveted Lap-joints

$$T = (p - d)\, t f_t. \tag{53}$$

$$S = \frac{2\, \pi d^2}{4} f_s = 1.57\, d^2 f_s. \tag{54}$$

$$C = 2\, dt f_c. \tag{55}$$

$$d = 1.27 \frac{t f_c}{f_s} \tag{56}$$

$$p = \frac{1.57\, d^2 f_s}{t f_t} + d. \tag{57}$$

For three or more rows of rivets, the formulas must be modified in a similar manner.

99. *Butt-joints* may have one or two cover-plates. In the former case cover-plates will have to have the same thickness as the plate. In the latter case each of the covers should have theoretically half the thickness of the plate. Experience shows, however, that with this thickness the cover-strap will usually fail before the main plate does, probably on account of unsymmetrical stresses, causing more than half the load to come on one of the cover-plates, and these straps are, therefore, generally made a little thicker than $\frac{t}{2}$.

When only one cover-plate is used, the rivets will be in single shear, and the formulas given for lap-joints may, therefore, be applied to this case. If there are two cover-plates, each rivet will be in double shear, and will offer practically twice the resistance to shearing that it would to single shear. The theoretical formulas for butt-joints with two cover-straps, each assumed to have a thickness of at least $\frac{t}{2}$, are, therefore, as follows:

$$S = \frac{2\,\pi d^2}{4} f_s' = 1.57\,d^2 f_s'. \tag{58}$$

$$T = (p - d)\,tf_t, \text{ as for lap-joints.} \tag{59}$$

$$C = 2\,dtf_c'. \tag{60}$$

Equating the values of S and C given in formulas (58) and (60), we obtain:

$$d = 0.64\,t\frac{f_c'}{f_s'}. \tag{61}$$

By equating T and S we find:

$$p = \frac{3.14\,d^2 f_s'}{tf_t} + d. \tag{62}$$

If the butt-joint is double-riveted, the formulas to use are:

$$T = (p - d)\,tf_t. \tag{63}$$

$$S = \frac{4\,\pi d^2}{4} f_s' = 3.14\,d^2 f_s'. \tag{64}$$

$$C = 2\,dtf_c'. \tag{65}$$

$$d = \frac{0.64\,f_c' t}{f_s'}. \tag{66}$$

$$p = \frac{3.14\,d^2 f_s'}{tf_t} + d. \tag{67}$$

100. Practical Considerations. — The theoretical formulas discussed above do not take into consideration the stresses caused in the rivets, when their shrinkage, due to cooling, is resisted by the plates. For small rivets these stresses may become excessive. In rivets having lengths of six or more inches, the shrinkage caused by cooling is apt to force off the rivet heads, unless the rivets are cooled at the center, before being driven. In actual practice the stock size of rivet that is nearest the diameter given by the theoretical formulas will be used, but there are limitations as to the size of rivets that can be driven. For hand-work the diameter of the rivet should be limited to $1\frac{1}{8}$ ins. For machine driving the diameter has been made as large as $1\frac{1}{2}$ ins. in special cases.

The formulas given above consider only the strength of a joint. In practice, we must, however, also secure joints that are tight, and this may oblige us to modify considerably the pitch given by theoretical formulas. In order to have sufficient room to make the rivet heads, there is, also, a limit to the shortness of the pitch. The results given by the theoretical formulas have, therefore, to be modified by the requirements established by experience.

The margin between the rivet-hole and the edge of the plate is usually made equal to the diameter of the rivet, or slightly more. With this width, riveted joints that have been tested to the breaking point have almost invariably failed by the shearing of the rivets, or the tearing of the plates between the rivet-holes. There are, therefore, two important ways of possible failure to which particular attention has to be paid in practice.

101. Empirical Formulas. — Numerous empirical formulas for rivet diameters and pitch have been proposed. The following formulas give the diameter of the rivets based upon the thickness of the plate.

Unwin gives $d = 1.2\sqrt{t}$ to $1.4\sqrt{t}$. (68)

Box gives $d = (1\tfrac{1}{4}\,t) + \tfrac{3}{16}$. (69)

Prof. Kennedy gives for lap-joints $d = 2\tfrac{1}{3}\,t$.

The diameter of the rivet should be taken to the nearest $\tfrac{1}{32}$ in.

Emil Kuichling, M. Am. Soc. C. E., discussed the subject of riveted joints very thoroughly, and gave both theoretical and practical formulas, the latter being based upon a study of more than twenty empirical formulas and tables.*

In using practical formulas for the diameter and pitch of rivets, we can no longer maintain the ideal requirement.

$$T = S = C.$$

The efficiency of the joint will be, therefore, the minimum value found by dividing T, S, and C by P.

The distance from the edge of the plate to the rivet hole, called the margin, cannot be determined by theoretical formulas. When wrought iron rivets are driven in steel plates, the margin is usually made equal to the diameter of the rivet. When steel rivets are used the margin is generally increased to $1.25\,d$.

The American Machinist, May 3, 1906, gives the following minimum distances from the center of any rivet-hole to the edge of the plate:

* "The Design of Riveted Steel Pipes," by Emil Kuichling, *The Polytechnic*, 1898 and 1899, XV, p. 125.

TABLE XXXIII. — MINIMUM MARGIN, CENTER OF RIVET TO EDGE OF PLATE

Diameter of rivet-hole	Sheared edges	Rolled edges
Inches	Inches	Inches
$\frac{7}{8}$	$1\frac{1}{2}$	$1\frac{1}{4}$
$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{1}{8}$
$\frac{5}{8}$	$1\frac{1}{8}$	1
$\frac{1}{2}$	1	$\frac{7}{8}$

The maximum distance from any edge should not exceed $8 \times t$.

For double chain-riveting the distance between the center lines of the rivets should not be more than 2.5 d. If the riveting is *staggered* the distance between the rows should be 1.88 d. This will insure safety against zigzag tearing, but it may bring the rivet heads too close together.

102. Relative Efficiencies of Various Kinds of Joints. — Stoney gives in his "Strength and Proportions of Riveted Joints" the following table of efficiencies of various kinds of riveted joints, as found by actual tests.

TABLE XXXIV. — RELATIVE EFFICIENCY OF WROUGHT IRON JOINTS

Description	Efficiency, per cent
Original solid plate	100
Lap-joint, single-riveted, punched	45
Lap-joint, single-riveted, drilled	50
Lap-joint, double-riveted	60
Butt-joint, single cover, single-riveted	45–50
Butt-joint, single cover, double-riveted	60
Butt-joint, double cover, single-riveted	55
Butt-joint, double cover, double-riveted	66

RELATIVE EFFICIENCY OF STEEL JOINTS

Description	Efficiency, per cent		
	Thickness of plates		
	$\frac{1}{4}$–$\frac{3}{8}$	$\frac{1}{2}$–$\frac{5}{8}$	$\frac{3}{4}$–$\frac{7}{8}$
Original solid plate	100	100	100
Lap-joint, single-riveted, punched	50	45	40
Lap-joint, single-riveted, drilled	55	50	45
Lap-joint, double-riveted, punched	75	70	65
Lap-joint, double-riveted, drilled	80	75	70
Butt-joint, double cover, single-riveted, drilled	70	65	60
Butt-joint, double cover, double-riveted, punched	75	70	65
Butt-joint, double cover, double-riveted, drilled	80	75	70

TABLE XXXV.—STRENGTH OF MATERIALS IN RIVETED JOINTS

Values of f_t, f_s, and f_c for different kinds of joints, based upon experiments	Iron			Steel		
Kind of joint	f_t	f_s	f_c	f_t	f_s	f_c
Lap-joint, single-riveted, punched holes	40,000	38,000	67,000	....	47,500	85,000
Lap-joint, single-riveted, drilled holes	45,000	36,000	67,000	....	45,000	85,000
Lap-joint, double-riveted, punched holes	45,000	40,000	67,000	....	48,000	85,000
Lap-joint, double-riveted, drilled holes	50,000	38,000	67,000	....	46,000	85,000
Butt-joints, single cover: Use values given for lap-joints		f_s'	f_c'		f_s'	f_c'
Butt-joints, double cover, single-riveted, punched holes	40,000	42,500	89,000		48,000	100,000
Butt-joints, double cover, single-riveted, drilled holes	45,000	41,000	89,000	....	46,000	100,000
Butt-joints, double cover, double-riveted, punched holes	45,000	38,000	89,000	....	47,500	100,000
Butt-joints, double cover, double-riveted, drilled holes	50,000	36,000	89,000	....	45,000	100,000
Original plate *	50,000	....	....	60,000	....	....
Original bar *	....	45,000	....	....	52,000	....

The Master Steam Boiler Makers' Association recommends, as the result of tests made by its committee: for iron rivets, $f_s = 42,000$ and $f_s' = 40,000$; for steel rivets, $f_s = 46,000$, $f_s' = 44,000$. They do not give f_t.

The Boiler Code Committee of the A. S. M. E. recommends: for iron rivets, $f_s = 38,000$ and $f_s' = 35,000$; for steel rivets, $f_s = 42,000$ and $f_s' = 39,000$. They recommend for maximum values: for iron rivets, $f_s = 38,000$, $f_s' = 38,000$; for steel rivets, $f_s = 44,000$ and $f_s' = 44,000$. They recommend $f_t = 55,000$ for mild steel and equals 45,000 for wrought iron, where actual tensile strength is not known. For compressive strength they recommend $f_c = 25,000$ for mild steel.

* If the original material varies from what is given in this table, the other values must be varied proportionately.

Table XXXV, reproduced from page 112 of Smith and Marx's treatise on "Machine Design," gives the average values of the different elements of strength in riveted joints.

103. English Practice in Riveted Joints.* — Spooner gives the following values for the strengths of steel plates, steel rivets, etc.:

	Tons per sq. inch
Tensile strength of unperforated steel boiler plate..........	28
Resistance of rivets to shearing..........................	23
Resistance of rivets to crushing..........................	46

If the diameter of a rivet is made less than the thickness of the plate, there is danger of the punch being crushed. For this reason The British Board of Trade has made the rule that the diameter of the rivet must not be less than the thickness of the plate. Empirical English formulas for this thickness were given on page 108.

In the following table will be found the size of rivets and minimum pitch recommended by the National Boiler Insurance Co. of England.

TABLE XXXVI. — SIZE OF RIVETS AND MINIMUM PITCH

Thickness of plate	Diameter of finished rivet	Minimum pitch	
		Single riveting	Double riveting
Inch	Inch	Inch	Inch
$\frac{3}{8}$	$\frac{3}{4}$	$1\frac{3}{4}$	2
$\frac{7}{16}$	$\frac{11}{16}$	$1\frac{7}{8}$	$2\frac{1}{8}$
$\frac{1}{2}$	$\frac{13}{16}$	2	$2\frac{1}{4}$
$\frac{9}{16}$	$\frac{13}{16}$	2	$2\frac{1}{4}$
$\frac{7}{8}$	$\frac{7}{8}$	$2\frac{1}{8}$	$2\frac{1}{2}$
$\frac{11}{16}$	$\frac{7}{8}$	$2\frac{1}{8}$	$2\frac{1}{2}$
$\frac{3}{4}$	$\frac{7}{8}$	$2\frac{1}{8}$	$2\frac{1}{2}$
$\frac{13}{16}$	$\frac{15}{16}$	$2\frac{1}{4}$	$2\frac{3}{4}$
$\frac{7}{8}$	$\frac{15}{16}$	$2\frac{1}{4}$	$2\frac{3}{4}$
$\frac{15}{16}$	$\frac{15}{16}$	$2\frac{1}{4}$	$2\frac{3}{4}$

The proportions of the rivet heads have not yet been standardized in England. Fig. 52, reproduced from Mr. Spooner's book on "Machine Design, Construction and Drawing," gives the usual English practice as regards rivet heads.

Two different kinds of tools are used for caulking the plates, *viz.*: the ordinary caulking steel (Fig. 53), having at its extremity a thickness of about $\frac{3}{16}$ in. and a width of $1\frac{1}{2}$ ins.; and the fullering tool, having a thickness equal to that of the plate (Fig. 54).

In boiler practice, the joints are usually caulked on the outside and

* Spooner's "Machine Design, Construction and Drawing," p. 142.

on the inside. This work is facilitated by shearing or grinding the edges of the plates to an angle of about 75 to 80 degrees. The tool burrs down the plate, so as to form a metal to metal joint. Care must be taken not to damage the plate below the tool, nor to spring the joint open. Fullering, which is largely superseding caulking in England, gives a clean finish and is less apt to injure the plate. Fig. 55 shows a combined fullering and caulking tool.

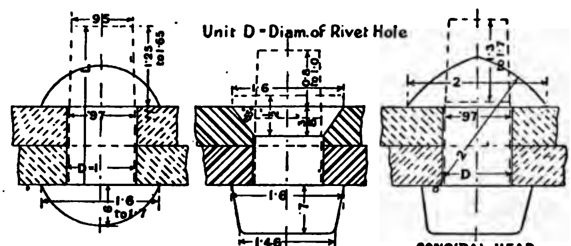


FIG. 52. — English Rivet Heads.

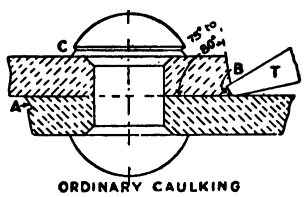


FIG. 53.

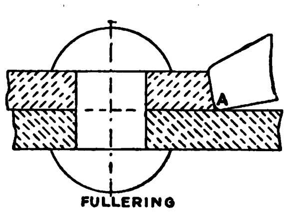


FIG. 54.

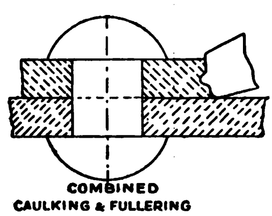


FIG. 55.

In girder work, $\frac{3}{4}$-in. rivets are generally used for plates less than $\frac{1}{2}$ in. thick, and $\frac{7}{8}$-in. rivets for $\frac{1}{2}$- to $\frac{3}{4}$-in. plates.

The British Board of Trade rule for distances between rows of rivets in zigzag work is:

$$\text{Distance between rows} = \frac{\sqrt{(11\,p + 4\,d)\,(p + 4\,d)}}{10}.$$

A rough rule makes the diagonal pitch equal to $\frac{p}{1.3}$.

The German Lloyd's rule for the diagonal pitch in zigzag riveting is:

$$\text{Diagonal pitch} = 2.4\,d.$$

Prof. Kennedy recommends:

$$\text{Diagonal pitch} = \frac{2}{3}p + \frac{d}{3}.$$

The margin between the rivet hole and the edge of the plate is made at least equal to the diameter of the hole, which makes the minimum lap of the plates 3 d.

In butt joints with two covers, each of the straps is made at least $\frac{5}{8}\,t$ in thickness.

For chain riveting, the distance between the pitch lines should be at least equal to 2 d. The British Board of Trade rule makes this distance (y):

$$y = 2\,d \text{ to } 2\,d + \tfrac{1}{2}.$$

The British Board of Trade rule is for double shear strength, which is assumed to be 1.75 × strength of single shear.

According to Mr. Spooner, well-designed joints for boiler work should have the following efficiencies:

Single-riveted joint, 50–55 per cent efficiency.
Double-riveted joint, 65–70 per cent efficiency.
Treble-riveted joint, 80–85 per cent efficiency.

104. Expansion and Contraction in Riveted Pipes. — In the first Rochester pipe (p. 324), provision for expansion and contraction was made by riveting a cast iron hub to one end, and a cast iron spigot to the other end, of each section of pipe. As these sections had usually a length of about 27 ft., and never over 81 ft., ample allowance for the effects caused by changes of temperature was thought to have been made. This arrangement did not, however, prove to be satisfactory. The joints were partly drawn out in winter, and did not assume their original good condition in summer. This caused considerable leakage at the circular joints of the pipe-line, and necessitated constant inspection. Similar experience was made in California.

In all recent lines of riveted pipe, no expansion joints are used, the metal of the pipe being considered strong enough to resist successfully stresses caused by changes of temperature. In such cases, however, the pipe must be well anchored at its ends in masonry, and, also, at any intermediate places where gate-houses, valves, etc., may be introduced.

In steel pipes that are exposed and subjected to great changes of temperature some kind of expansion joint must be used. It may be simply a stuffing-box. Such a joint was placed at each end of the wrought iron pipe, $90\frac{1}{2}$ ins. in diameter and 1380 ft. long, which was constructed in a vault on "High Bridge" in 1863 to convey the supply of the Old Croton Aqueduct across the Harlem River to the City of New York. Other types of expansion joints are shown in Figs. 56, 57, and 58. The last is used extensively in France.

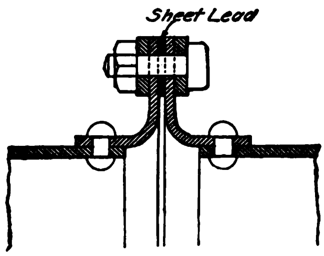


FIG. 56.

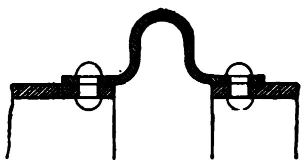


FIG. 57.

Expansion Joints.

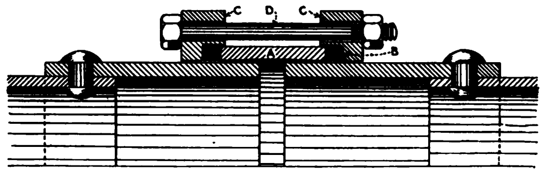


FIG. 58. — French Expansion Joint. *A*, *C*, and *D* are of steel; *B* is of rubber.

105. Spiral riveted steel pipe * (Fig. 59) is used extensively in water works and hydro-electric plants for pressures up to 400 lbs. per square inch. It is made from strips of sheet steel, which are wound into a helical shape with one edge overlapping the other, forming a lap-joint, which is secured by rivets. The outside lap is slightly offset, so that besides giving a good metal contact, it also makes the pipes smoother on the inside. The rivets are driven cold by compression, and fill the rivet holes completely and with a slight countersink. The pipe is made by machines from which it issues in a continuous piece, which is cut to any desired lengths. The pipe is manufactured in sizes of 3 to 40 ins. in diameter, usually in lengths of about 20 to 30 ft. and is immersed in a bath of asphalt, kept at a temperature of 400° F., to protect it from corrosion.

FIG. 59. — Spiral Riveted Pipe.

* Manufactured by the American Spiral Pipe Works, Chicago; and The Abendroth & Root Mfg. Co., New York.

The pipes are provided with forged steel flanges which are riveted to the ends of the pipes, and the flanges of adjoining pipes are bolted together. Where expansion joints are required, they are made as shown in Fig. 60. For submerged pipes the flexible joint shown in Fig. 61 is used. Specials for spiral riveted pipes are shown in Fig. 62.

Spiral riveted pipe is stronger than pipe of the same thickness of metal made in the ordinary manner with horizontal and circumferential joints. The spiral seam is the strongest part of the pipe. This type of pipe has the additional advantage of lightness, and for this reason is often used in mountainous regions, where transportation is difficult. The pipes are easily put together.

Spiral riveted pipe has been used in the water works of Portsmouth and Gorham, N. H., Brattleboro, Vt., Langerville, Dixfield, Millinocket and Lewisport, Maine.

FIG. 60. — Expansion Joint.

FIG. 61. — Flexible Pipe-joint.

Among some of the western lines of spiral riveted pipes may be mentioned:

TABLE XXXVII. — SPIRAL RIVETED PIPES

Owner	Location	Length	Diameter	Maximum head
		Feet	Inches	
Cairo Contracting & Bridging Co.	Cairo, Ill.	6,600	12	....
....	Newaygo Co., Michigan	....	10	231
....	Osborn Creek, Nome, Alaska	....	24	....
Newhouse Mines & Smelting Co.	Salt Lake City, Utah	44,000	12 and 14	577
Beaver River Power Co.....	Utah	12,000	30, 32, and 34	191
Homestake Mining Co......	Lead, So. Dakota	17,000	28 and 26	462

106. Lap-welded steel pipe * is made of low carbon steel plates of the best quality, in sizes of 1½ ins. to 30 ins. diameter. Great care is

* Manufactured by the National Tube Company, Pittsburgh, Pa., Amer. Spiral Pipe Works, Chicago, Spang Chalfant & Co., Pittsburgh, Pa., and A. M. Byers & Co., Pittsburgh, Pa.

taken to secure a uniform mixture of metal in the blast furnace, by using high-grade ore, and combining the molten iron from several blast furnaces in one large 300-ton mixer. The metal is then refined in a converter into low carbon steel, and cast into ingots. These are reheated

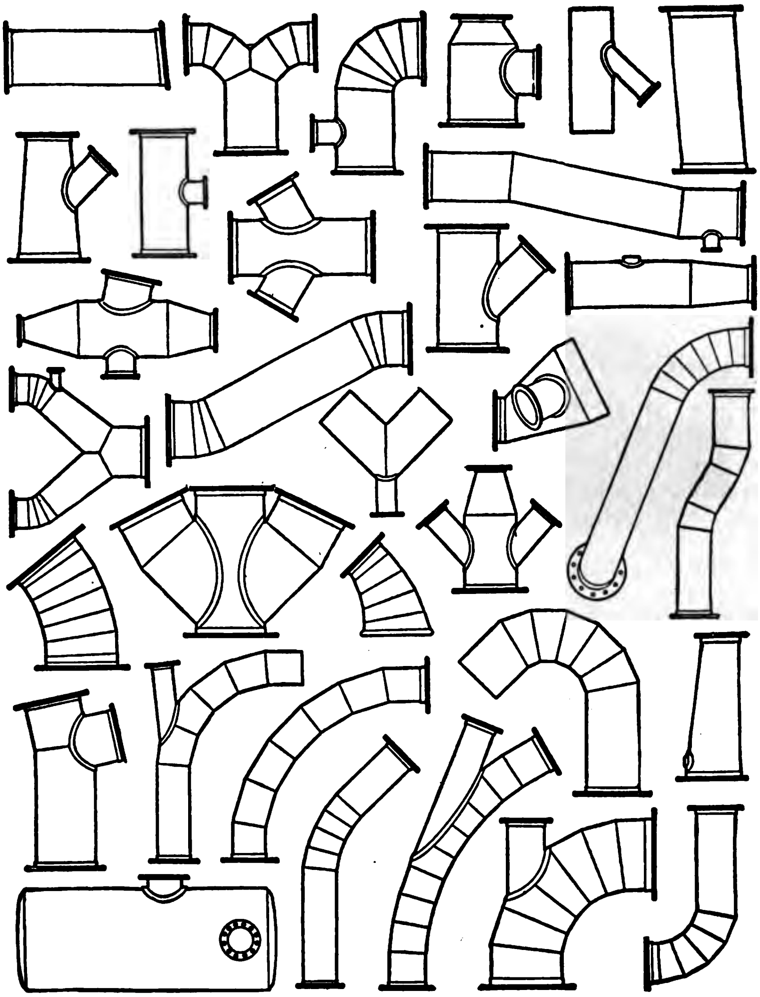


Fig. 62. — Specials for Spiral Riveted Pipes.

and rolled into blooms, which, after a second reheating, are rolled into plates of the required thickness and width, suitable for making the various sizes of pipe. The plates are bent, with the edges overlapping, and are then heated to a high temperature in a welding furnace, and passed over a pointed cylindrical mandril between two rapidly revolving rolls. The mandril is held in such a way, that it acts like an anvil, on which the overlapping edges of the tube are forced together and welded, by the pressure of the rolls. The tube goes next to a table, on which it is kept rolling, to maintain its circular form, and it straightens, while cooling. Both ends are then cut square, and one end is heated and expanded to form a bell end.

Lap-welded pipe comes in random lengths of 17 to 20 ft., the average length being about 18 ft. With this length, approximately 280 joints occur in a mile of pipe, instead of 440 joints, the number required per mile with ordinary cast iron pipes. There is, also, with lap-welded pipes, a saving as regards the quantity of lead required per mile. Lap-welded pipes, like spiral riveted pipes, have the advantages of light weight. They can be made of plate up to 1 in. in thickness. The pipes are dipped into a bath of hot asphalt, or, first galvanized and then dipped into the bath.

FIG. 63. — Flange-joint.

FIG. 64. — Stove-pipe Joint.

FIG. 65. — Pipe-joint for Low Pressure.

107. Forge-welded steel plate pipes are made in Germany in lengths up to 60 ft., and have been used in many places. In the power plant at Necaxa, Mexico,* 30-in. forge-welded pipes were used under a maximum head of 1450 ft. The pipe was made in Germany in 30-ft. sections, each of which was forged complete with flanges from one sheet of steel. Rubber gaskets were placed in the joints.

108. *American manufacturers* † have recently commenced to make forge-welded pipes by processes similar to those used in Germany, and are prepared to manufacture such pipes up to 8 ft. in diameter of plates up to $1\frac{1}{2}$ ins. in thickness.

109. Joints for lap-welded and forge-welded steel pipes are shown in Figs. 63, 64, and 65. The first of these joints is the ordinary forged steel flange connection; the second can be used where field riveting is possible. The *flared end* makes it easy to insert the taper end of the

* Trans. Am. Soc. C. E., 1907, LVIII, p. 42.

† American Spiral Pipe Works, Chicago, M. W. Kellogg & Co., New York, and Continental Iron Works, Brooklyn, N. Y.

adjoining pipe, and avoids a diminution of the diameter of the pipe by projecting rivet heads. Fig. 65 is used for low pressures and requires less lead than the quantity used in cast iron bell-spigot pipes. The *bolted-socket-joint* which allows a slight deflection at each joint is especially suited for long lines of pipe or for connections or submerged pipe-line. The *standard bolted expansion joint*, illustrated in Fig. 60, p. 115, can, also, be used with lap-welded or forge-welded pipe.

For high-pressure works, the ends of the pipes are heated and turned over the hubs of the flanges (Fig. 66), which makes leakage impossible, except through the gasket. With this joint, the flanges are left loose and can be turned so as to bring the bolt holes in line.

Another type of joint (Fig. 67), which is much used by the U. S. Navy, is made by boring a forged steel flange a little smaller than the outside

FIG. 66. — Pipe-joint for High Pressure.

FIG. 67. — Pipe-joint with End of Pipes Peened into the Flange.

of the pipe, and then shrinking it, while hot, on the pipe. The end of the pipe is usually peened into the flange by an expanding machine or by hammering.

The National Tube Company of Pittsburgh, Pa., uses with its lap-welded pipes the *Matheson joint* (Fig. 68), which is a bell-and-spigot joint, very similar in appearance to that of a cast iron pipe-joint. In the Matheson joint the spigot end of the pipe has a groove cut in it, corresponding to a recess in the bell, which acts with it as a lock, when the lead is poured and caulked. These joints are used for pipes of 2 to 30 ins. in diameter and 18 to 20 ft. long.

The Matheson joint is so designed that inside the pipe there is a continuous, straight, smooth surface, which reduces friction losses to a minimum. Another advantage of this joint is that small variations in alignment and grades are taken care of in the joint. By using short lengths of pipe, expensive fittings and pipe bends can be omitted, the deflections in the line being made at the Matheson joint.

110. Failure of Lap-welded Pipe. — Some failures of large lap-welded pipes, due to imperfect welding, have occurred. J. D. Galloway,* M. Am. Soc. C. E., mentions a 30-in. pipe of this kind, in which one of the seams opened, shortly after the pipe was installed, and caused much damage. After being in use for seven years, another seam opened. This was found to be caused by a very imperfect weld. There was, also, at this joint, a shearing of the outer portion of the steel from the inner portion, along the neutral axis. The steel appeared to be in layers. In another line of 30-in. lap-welded pipes, the welds were discovered to be defective in several places. Two defective welds were found in a line of 24-in. lap-welded pipe, one before the pipe was laid, and another, two weeks after the pipe-line had been put in service. Such failures are practically unknown with modern lap-welded pipes.

FIG. 68. — The Matheson Joint.

111. Lock-bar pipe † (Fig. 69) is made of plates of steel or ingot iron, in thicknesses of $\frac{3}{16}$ to $\frac{1}{2}$ in., the longitudinal seams being joined by means of a *lock-bar* extending the whole length of the pipe, instead of being joined by rivets. Each length of pipe consists of two metal plates, curved to a semi-circle, having their longitudinal edges upset so as to fit the lock-bar. When the plates have been assembled, a lock-bar of steel is forced by hydraulic pressure over each set of longitudinal edges, forming thus the lock-bar joint. This type of joint provides a metal pipe the strength of which is limited only by the strength of the plate itself.

Lock-bar pipe was used for the first time in 1898 and 1899 in the Coolgardie pipe-line, in Western Australia, which is 30 ins. in diameter and 325 miles long (p. 338). In 1905 the City of Lynchburg, Va., made the

* Trans. Am. Soc. C. E., 1907, LVIII, p. 59.

† Letters patent for lock-bar pipe were granted to Mephan Ferguson, of Perth, Australia, in Great Britain, Germany, France, and the United States. The patent and exclusive rights for its manufacture and sale in the United States and Canada have been acquired by the East Jersey Pipe Corporation, Paterson, N. J.

first use of this pipe in the United States, when it laid 15,000 ft. of 30-in. lock-bar pipe. In that year it was, also, installed in Wilmington, Del., Pittsburgh, Pa., and Paterson, N. J. Since that time, it has been used by many other cities in the United States and Canada for water supply and distribution. It has, also, obtained considerable usage as penstock for high-pressure hydraulic installations.

112. Method of Manufacture. — This type of pipe can be made much more rapidly than riveted pipe of the same size, as all punching, drilling, reaming, riveting, and caulking on the longitudinal seams is avoided. The flat sheets of steel, in standard lengths of about 30 ft., are taken direct from the cars by cranes, and placed singly on the laying-out floor. After the plates have been marked off to dimensions, and the rivet-holes at the ends have been properly spaced, the plates are

Fig. 69. — Lock-bar Pipe.

carried by electric cranes to the multiple punches where the rivet-holes on the ends of the plates for the circumferential joints are cut. The plate is then transferred to the bed of the upsetting machine, one end being held firmly by a movable plate-girder clamp, with a vertical web, which is operated by hydraulic jacks. A traveling planer-head, with fine horizontal planing tools, moving parallel with the clamp, trims the edge of the steel plate to the exact size. The planer is followed by a set of six to eight vertical grooved rollers, 3 ins. in diameter, that act successively on the edge of the steel plates, upsetting the metal in proportion to the thickness of the plate, the upset forming a shoulder that in a later process is firmly engaged by the lock-bar. When one edge has been upset, the sheet is turned around, and its other edge is treated similarly.

The plates are then placed in the crimping machine, a hydraulic press of 150 tons capacity, with dies 4 to 8 ft. long, to curve the edge of the plate to the required radius. This is done to prevent injury to the upset edge in the bending rolls, to which the plate goes next, and from which it comes out curved to a semicircle. The curved plates are then taken to the assembling floor, where two plates are matched together, and their edges fitted with two lock-bars. They are held firmly in that position by solid steel rings, about 5 ft. apart, which enclose the two plates and are wedged tight against the bars.

The plates, thus assembled, are delivered to the lock-bar press, which is a 750-ton hydraulic cylinder, set vertically under an adjustable cross-head, on heavy steel bolts. The cross-head is set to provide clearance between it and the plunger for the pipe, and a pair of adjustable die-blocks, mounted on a long, horizontal steel arm, and operated by a small vertical hydraulic cylinder, which is mounted in the pipe. The plunger of the small cylinder is extended so that the die-blocks engage the inner faces of the two lock-bars and act as a holder-on. The large hydraulic cylinder outside is then operated, and reacting through the inner cylinder against the dies of the cross-head, closes the open flanges of the lock-bars, pressing them solidly down on the upset edges of the covered plates, in lengths of 6 ins. on thick plates to 12 ins. on $\frac{3}{16}$- and $\frac{1}{4}$-in. plates. The pipe is then placed on the trimming skids, the ends of the lock-bars that project are cut off, and the bars are chamfered down at the ends, to permit a snug fit for the circular field joints. For this reason, the inner faces of the bar are also flattened, affording at the same time bearing for a rivet.

Each pipe is tested to at least one and one-half times its intended working pressure in a large adjustable testing machine, having heavy disced heads with rubber gaskets. The pipe is then filled with water from a pump capable of exerting the desired pressure, which is maintained while a careful inspection is made of the lock-bar joints.

After being tested, the pipe is thoroughly cleaned and dried. It is then heated uniformly in a large oven to a temperature of 350° F., and dipped vertically in a bath of mineral rubber coating, maintained at a temperature of 350° F., and of sufficient depth to allow the pipe to be entirely submerged. After the pipe sections have been removed from the bath, they are set in a vertical position while cooling. When the pipe has been dipped in the mineral rubber coating, and the coating is sufficiently set to prevent flow in the subsequent operations, it may, as an extra precaution, optional with the purchaser, be wrapped with 10-ounce Calcutta burlap, cut into strips 18 ins. wide, and applied on a spiral wrapping machine.

The dipping tank is made of two concentric cylinders, 10 ft. and 12 ft. respectively in diameter, set vertically on a concrete base, the space between the two cylinders being occupied by steam coils. The inner face of the inner cylinder also has a set of heating pipes, and, on the axis of the cylinder, stands a 12-in. pipe that contains an auxiliary heating system, which insures an even temperature to the bath.

Lock-bar pipe is manufactured by the East Jersey Pipe Corporation of Paterson, N. J., in sizes from 20 to 72 ins. in diameter. On account of the absence of rivet heads along the longitudinal joints, and the omission of circumferential rivet seams, except on the ends, an increased carrying capacity of from 10 to 20 per cent is attained by this pipe over riveted pipe of the same size. The strength of the longitudinal joint

FIG. 70. — Lock-bar Pipe, Tested to Rupture.

has been proved by tensile tests and by hydraulic tests to be stronger than the plate itself, as will be seen in Fig. 70, which shows the result of an actual test.

For diameters of 24 ins. and upwards, which are sufficiently large for a man to work on the interior, riveted taper-joints are employed for ordinary purposes. This joint is similar to the *stove-pipe joint*, the end of one pipe fitting snugly into the end of the contiguous pipe, the circumferential seam then being riveted, and the edges of the plates being caulked both inside and out. Flanged joints, butt-strap joints, and special Custer (Fig. 71) are also used, depending on field conditions.

FIG. 71. — Custer Joint.

113. Cement-lined-and-coated Pipes. — Jonathan Ball invented a process of lining sheet-iron water pipes on the inside, and coating them on the outside, with hydraulic cement mortar, in order to prevent the iron from being corroded. Pipes of this kind were laid in Saratoga, N. Y., for domestic water supply as early as 1845.* Many hundred miles of this kind of pipe have been laid, especially in the New England

* J. T. Fanning's "Treatise on Hydraulic and Water-supply Engineering," 9th edition, p. 479.

States. During the Civil War cement-lined wrought iron pipes could be manufactured and laid more cheaply than cast iron pipes, and this consideration promoted their introduction, but of late years they have been less in favor.

This type of pipe is made either according to the *Goodhue and Birnie* or the *Phipps* patent.* In the first method, sheets of wrought iron of proper thickness to withstand the water pressure are single-riveted, cold, into cylindrical forms, without any attempt being made to secure water-tight joints. The pipes are then lined on the inside with $\frac{3}{4}$ to 1 in. of neat hydraulic cement or cement mortar, mixed 1 part cement to 1 part of sand. The pipes are laid in the trench on a bed of, and covered by, $\frac{3}{4}$ to 1 in. cement mortar. They are made in 9-ft. lengths, fastened together by means of wrought iron sleeves, and the joint cased, inside and outside, with cement mortar, or the joints may be telescoped together in stove-pipe fashion, and protected by a coating of cement mortar. Cast iron joints have been also used for cement-lined pipes, but have not proved to be very satisfactory.

In the second method, the pipes are made in a similar way to that described above, with this difference, however, that the pipes are coated, both on the inside and on the outside, before being laid in the trench.

Compared with cast iron pipes, cement-lined wrought iron pipes have the following advantages: (1) Freedom from tuberculation; (2) longevity and durability, under favorable conditions.

On the other hand, this kind of pipe has the following disadvantages: (1) The necessity of great care in the manufacture and laying of the pipe; (2) small factor of safety, and consequently liability to be injured by water-hammer; (3) difficulty of securing water-tight joints; (4) difficulty of making service-pipe connection; (5) difficulty of making repairs.

On account of these disadvantages, cement-lined wrought iron pipes have not proved generally satisfactory for distributing systems of pipe, but they can be used advantageously for supply pipes that are not subjected to greater pressure than about 65 lbs. per square inch.

In Plymouth, Mass., however, where the water pressure is only 30 to 70 lbs. per square inch, cement-lined water pipes have been in use since 1855.† The original pipe, of which a large amount is still in use, consisted of a sheet-iron shell, about 9 ft. long, lined on the inside with about $\frac{1}{2}$ in. of cement mortar, mixed 1 to 1. The pipe was laid in the trench on a bed of mortar, and was covered with mortar. The pipes were butted together, and the joints were covered with steel sleeves, or collars, which were afterwards encased in mortar. Some of these pipes,

* Jour. New England Water Works Assoc., March, 1909, p. 3.

† *Eng. News*, Feb. 17, 1916, p. 300.

which were dug up, after being in continuous use for 60 years, were found to be in excellent condition.

Plymouth manufactures its own cement-lined pipes. No difficulties are said to be experienced there in tapping the pipes, which is done in the following manner: the jacket is cut away with a cold chisel, and the intervening cement is chipped away down to the shell of the pipe, which is then tapped in the usual way.

In 1914 the city had 55¼ miles of cement-lined pipes, 2 to 24 ins. in diameter. At that time the cost of the pipes was about the same as of those made of cast iron. The annual repairs amounted to $2.20 per mile, and the leakage averaged only 1 leak to 4 miles of pipe.

CHAPTER VII

PIPES OF VITRIFIED STONE-WARE, CEMENT, AND CONCRETE

114. Vitrified Stone-ware Pipe. — Terra-cotta pipes have been used from the remotest periods for conveying water. In the ruins of Ur of the Chaldees, Nineveh, Troy, Jerusalem, Pompeii and Herculaneum, etc., such pipes, laid thousands of years ago, are still found in good condition. They have not, however, much strength, and were encased with masonry, when used for water under pressure.

In modern water works, salt glazed vitrified stone-ware pipe has been used in a few cases for conveying water, not under pressure. Such pipes, 12 to 18 ins. in diameter, were used in the conduits of Amsterdam, Little Falls, and Johnstown, N. Y.* They were made with hub-and-spigot joints, which were filled with Portland cement mortar. The pipes were made extra thick, in order to withstand the pressure of the refilling.

The cheapness of vitrified terra-cotta pipes, their durability, and the fact that they are not subject to tuberculation may make it advisable to use them for conveying water which is not subject to pressure, providing the diameter of the pipe does not exceed 24 to 30 ins.

115. Cement-mortar pipes, 6 to 36 inches in diameter, have been used extensively in some of the Western States, in connection with irrigation works.† These pipes are made in lengths of 2 ft. One end of the pipe tapers inward and the other tapers outward, to form a beveled lap-joint, when abutting ends are connected together with cement mortar.

The pipes are made of a moist mixture of cement and sand which is tamped into metal molds. For pipes up to 18 ins. in diameter, the mixture consists usually of 1 part cement to 4 parts of sand. For larger pipes, 1 part cement is mixed with 3 parts of sand. The molds are removed as soon as the tamping has been completed. The pipes are carefully cured, by being kept moist for nearly a week, and are allowed to harden for a month, before being laid.

The strength of a cement pipe depends as much upon the joints as upon the pipe itself. Pipes made of moist mortar are more porous and weaker than those made of wet mortar, and they are liable to be cracked

* *Eng. Record,* May 26, 1888, p. 361.

† "Irrigation Practice and Engineering," by B. A. Etcheverry, Vol. II, p. 301, New York, 1915.

by expansion and contraction. When the pipes are used as laterals of an irrigation system, cracks are not a serious objection to the pipes being used.

The following table* shows what heads can be safely borne by cement mortar pipes. The bursting values are about four times the values given in the table.

TABLE XXXVIII. — SAFE PRESSURE HEADS FOR HAND-TAMPED CEMENT PIPE

Diameter of pipe in inches	Maximum safe head in feet		
	For 1 to 2 mixture	For 1 to 3 mixture	For 1 to 4 mixture
12	20	15	10
14	20	15	10
16	18	12	8
18	18	12	8
20	18	12	8
24	15	10	6
30	15	10	6

116. Concrete pipes are made in a similar manner as cement-pipes, 1 part of cement being mixed with 4 parts of natural aggregate of pit gravel and sand, for pipes up to 18 ins. in diameter. For larger pipes, 1 part of cement is mixed with 3 parts of pit gravel and sand. If screened gravel or crushed rock is used, the mixture should be about 1 part of cement, 2 parts of sand, and 3 or 4 parts of gravel rock, which should not be larger than ½ in. in diameter.

The pipes are usually coated on the inner surface with a thin paste of neat cement, in order to make them less permeable. To insure water-tightness, hydrated lime, equal in weight to 5 per cent of that of the cement, is often added to the concrete mixture.

The tamping of the concrete into molds is usually done by hand, but machinery has, also, been employed for this purpose. In the latter case, the pipes are generally made with bell-and-spigot joints.

117. Reinforced Concrete Pipe. — For diameters larger than 3 ft., concrete pipes are usually reinforced with steel bars. These bars are made of sufficient size to resist the whole water pressure and any water-hammer that may occur. The concrete is made only thick enough to prevent percolation. When the pipes are to be subjected to a considerable pressure, they are sometimes provided with a lining of steel plate, placed either at the inner surface of the pipe, or embedded in the concrete.

* "Irrigation Practice and Engineering," by B. A. Etcheverry, Vol. II, p. 303.

Pipes of reinforced concrete can either be made in sections, 4 to 8 ft. long, which are lowered into the trench, or they can be constructed in the trench as a continuous pipe. The former method can only be employed for pipe up to about 6 ft. in diameter. The circumferential reinforcement, which has to resist the water pressure and water-hammer, may consist of bands, placed at the required intervals; or it may be formed, for pipes not larger than about 4 ft. in diameter, of a band, wound spirally by machinery, as is done for wood-stave pipes. The longitudinal reinforcement serves to keep the circumferential reinforcement in place, and to resist temperature stresses. If expansion joints are provided, the longitudinal reinforcement can be made very light.

To insure water-tightness, the concrete, for heads of less than 50 ft., should be mixed in the proportion of 1 part cement, 2 parts of sand, and 4 parts of stone. For heads of 50–100 ft., a 1 : 2 : 3 mixture of concrete should be employed.*

118. Lock-joint Pipe. — Pipes of concrete, reinforced with wire fabric, made according to the patented Meriwether system, are manufactured by the Lock-joint Pipe Company of New York. The pipes are either made in the trench, or cast in 4-ft. sections and lowered into the trench. The joints of the pipe are formed as shown in Fig. 72. The reinforcement of two adjoining pipe sections overlap in a recess, $1\frac{3}{8}$ to $1\frac{7}{8}$ ins. long, which is filled with cement mortar, poured through a hole at the top of the recess, thus locking the sections together and sealing the joint by one operation. For a 24-in. pipe, the thickness of the concrete is usually 3 ins. For a 7-ft. pipe it is about 8 ins.

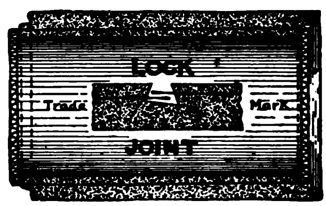


FIG. 72. — Lock-joint Pipe.

The following companies, also, are engaged in the manufacture of reinforced concrete pipe, *viz.:* The Reinforced Concrete Pipe Company of Los Angeles, Cal., and the Reinforced Pipe Company of Jackson, Mich.

119. Inverted siphons of reinforced concrete have been built in a number of places. Two important works of this kind, known as the Sosa Siphon and the Albelda Siphon, were built in the province of Huena, Spain.†

120. *The Sosa Siphon* consists of twin pipes of reinforced concrete, 12.47 ft. in diameter, and 3340 ft. long. It is subjected to a maximum

* For a full account of the making of reinforced concrete pipe see Etcheverry's "Irrigation Practice and Engineering," Vol. II, pp. 310–338.

† *Ibid.*, p. 333.

head of 85 ft. The twin pipes consist of 158 sections, each 21.32 ft. long, formed of a steel tube, $\frac{1}{8}$ in. thick, on the outside of which are spaced hoops of T-bar reinforcement, which are surrounded with, and embedded in, a concrete shell, 5.9 ins. thick. The pipes are lined on the inner surface with reinforced mortar, 0.87 in. thick. The ends of the steel tubes were connected by a riveted steel collar, shaped with a corrugation, to allow for expansion and contraction. This collar was well coated with a hot mixture of tar and asphalt. A collar of reinforced concrete was placed over each joint. At the crossing of the Sosa River, the siphon pipes are carried by an arched bridge of concrete.

121. *The Albelda Siphon*, built in 1909, is formed of a single pipe of reinforced concrete, 13.12 ft. in diameter, and 2363 ft. long. It is sub-

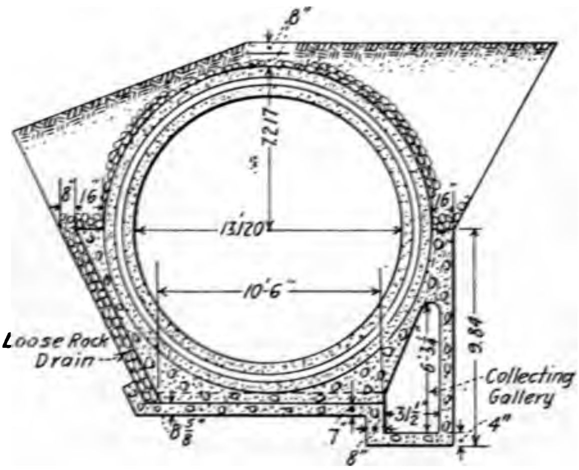


FIG. 73. — Albelda Siphon, Spain.

jected to a maximum head of 97 ft. This siphon has no expansion joints, and no steel tube, to insure water-tightness. The siphon is 7.87 ins. thick, and is supported, up to its horizontal diameter, by a concrete cradle. As some leakage was expected from the siphon, the cradle was provided with a drainage system (Fig. 73), and was made of very porous concrete, to enable the water leaking from the siphon to find its way into the drains.

The shell of the pipe, in which the reinforcement is placed, is 7.28 ins. thick. It is plastered on the inside with $\frac{1}{2}$ in. of cement mortar, making the total thickness of the pipe 7.87 ins. The reinforcement consists of 124 longitudinal round rods, spaced about 4 ins. apart, and of circumferential T-bars, tied at their intersection to the longitudinal rods with wire, about $\frac{1}{16}$ in. in diameter. Each of the T-bars has an exterior diameter of 13.4 ft., and is formed of two halves, butt-joined together with six rivets. The concrete for the conduit was composed of 1 part of Portland cement, 1.28 parts of sand, and 2.56 parts of gravel, less than $1\frac{1}{4}$ ins. in diameter. The interior plastering was made of equal parts of cement and sand.

122. *Siphons of the United States Reclamation Service.*—The first siphon of reinforced concrete for the Umatilla Project was completed in 1908. It is 47 ins. in inner diameter, and 468 ft. long. The pipe was made in sections, 8 ft. long and $2\frac{1}{2}$ ins. thick. The sections were cast vertically, by placing a steel core between the forms, and filling the molds with wet concrete. The reinforcement consists of round wire, which is wound into spirals, 4 ft. long, two being used for each 8-ft. section. After the spiral is wound, the proper spacing is maintained by longitudinal rods, 4 ft. long, which are tied to the spiral with wire. Cross-lacing is used, to give additional rigidity to the spiral. The sections of pipe were kept moist for 10 days, after being cast, and were allowed to harden for about 30 days, before being transported and laid.

This pipe-line, which is one of the first of its kind in America, was laid in sandy soil, which drains readily. It is subjected to a maximum head of 55 ft. Although exposed to temperatures as low as 29° F., the siphon has proved to be satisfactory in every respect. Tests of water-tightness, made soon after its completion, showed a total leakage of only $\frac{1}{2}$ gallon per second.

Another siphon on the Umatilla Project is 46 ins. in inner diameter, and 4700 ft. long. The maximum head on this pipe is 55 ft. The pipes were cast in 8-ft. sections near the trench. The reinforcement consists of $\frac{5}{16}$ in. wire of mild steel, wound on a drum into a helical coil, spaced, according to the head, to give a unit stress of 12,000 lbs. per square inch on the steel. The joints are covered by collars, 3 ins. wide and 3 ins. thick. Tests made by the U. S. Reclamation Service on a 46-in. pipe of reinforced concrete, under a head of 110 ft., showed that the pipe could stand a pressure of 50 lbs. per square in., without excessive leakage.

The Belle Fourche River siphon has an inner diameter of 5 ft., and a length of 3565 ft. It is subjected to a maximum head of 65 ft.

The Anderson siphon (Fig. 74) has an inner diameter of 8 ft., and a length of 477 ft. The maximum head on this siphon is about 70 ft.

The Whitewood siphon has an inside diameter of 6 ft., and a length of 395 ft. The maximum head on this siphon is only 16 ft.

All of these siphons have shells of reinforced concrete, 8 ins. thick. The circumferential reinforcement consists of twisted $\frac{1}{2}$- and $\frac{5}{8}$-in. bars, spaced $4\frac{1}{4}$ to 12 ins. apart between centers, according to the pressure that has to be resisted. The ends of the bars were securely fastened by welding. The longitudinal reinforcement of the three siphons is formed of $\frac{1}{2}$-in. twisted steel bars, spaced about 12 ins. apart. These bars overlap 20 ins. at the ends and are tied with wire. All intersections are wrapped with wire.

The concrete was mixed wet by machinery in the proportion 1: $2\frac{1}{4}$, $3\frac{3}{4}$, with 20–22 per cent of water. The gravel and crushed stone used are $\frac{1}{4}$ to 1 in. in diameter.

123. Aqueducts of Reinforced Concrete.* — Portions of the Los Angeles Aqueduct, adjoining siphons of steel pipe, were constructed of reinforced concrete. These sections have inner diameters of 10 feet,

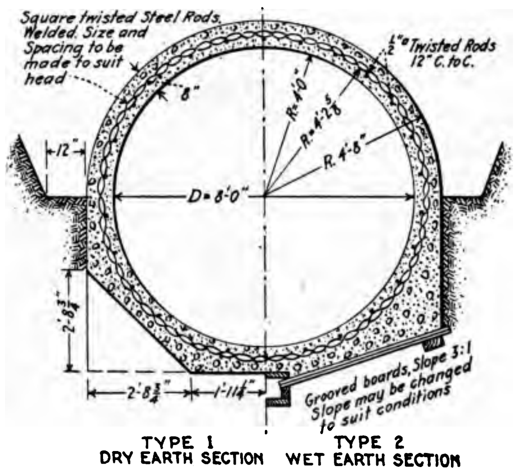


FIG. 74. — Anderson Siphon.

and are subjected to heads of 40 to 75 ft. For greater heads, steel pipes were found to be cheaper than those made of reinforced concrete.

The mixture of concrete used for the sections of reinforced concrete consisted of 1 part of cement, 2 parts of sand, and 4 parts of stone. Tufa cement was found to make denser concrete than ordinary Portland cement, as it is ground finer. The shell of the aqueduct is 9 ins. thick on the top and sides and the bottom is supported by a cradle of concrete. Fig. 75 shows one of these pipes. Under a head of 70 ft., the reinforced concrete is stressed 200 lbs. per square inch, a stress which good concrete, one month old, is able to withstand, without the assistance of the steel.

The steel reinforcement rods were placed $\frac{1}{3}$ in. from the outside edge

* "Final Report on the Construction of the Los Angeles Aqueduct," 1916, p. 209.

of the pipe. They were lapped 18 ins., and wired together. The details of the reinforcing is given in Fig. 75.

The pipes were all buried in trenches and covered with soil. The back filling was wetted down during the curing of the pipe, and the ends of the pipe were closed with curtains, in order to maintain a humid atmosphere in the pipes. After a pipe was 4 to 6 weeks old, it was filled slowly with water, and kept full.

Transverse Reinforcement Spaced 4 ins. Center to Center.

Head	Rods	Length
15′	3/8″	35′ 4″
30′	1/2″	35′ 10″
45′	5/8″	36′ 5″
70′	3/4″	36′ 11″

Rods ½ in. from outer edge of pipe.

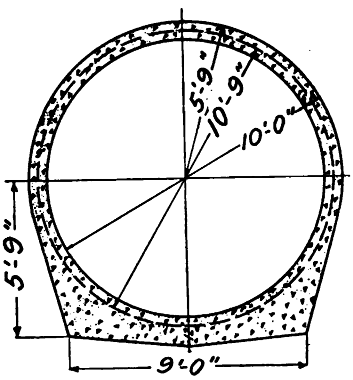


FIG. 75. — Reinforced Concrete Conduit, on Los Angeles Aqueduct, Cal.

In the first two concrete pipes, expansion joints of the Z-type were used, the joints being filled with asphalt. No expansion joints were placed in the other concrete pipes, and no trouble resulted from this omission.

On the Catskill aqueduct, built recently to supply New York with water, certain pressure sections, 7.75 to 17 ft. in diameter, subjected to heads of less than 50 ft., are built of reinforced concrete.*

* "Water-works Hand-book," by Flinn, Weston, and Bogert, p. 374.

CHAPTER VIII

STRESSES IN WATER PIPES

124. Stresses in Water Pipes. — All kinds of water pipes are subjected to certain stresses, caused by the pressure of the water in the pipe, the weight of the refilling over the pipe, water-hammer, resulting from closing gates quickly, and the effect of changes of temperature. These stresses are usually neutralized by the resistance of the material in the pipe-shell. On curves and steep grades unbalanced forces may exist, which may necessitate the construction of anchorages, to keep the pipe from being displaced. A water pipe is usually supported for its whole length by the refilling which has been rammed around it. The refilling gives the pipe a certain amount of stability to resist any tendency to collapse, and opposes, also, the motion due to expansion and contraction. If the pipe is not supported for its whole length — as when it rests on masonry piers — it will act as a beam, causing thus longitudinal stresses.

125. Stresses due to Water Pressure. — The stresses caused by the outward pressure of the water are readily calculated. As the thickness of the pipes is in all ordinary cases only a small fraction of the diameter, the pipe may be considered to be a *thin ring*, and its thickness can be calculated by the well-known formula:

$$s = \frac{pr}{t}, \tag{70}$$

where s = stress per square inch in the iron of the pipe;
p = pressure per square inch of the water;
r = radius of the pipe in inches;
t = thickness of the pipe in inches.

Formula (70) is based upon the assumption that the material in the walls of the pipe is devoid of elasticity, and that the stress is the same on all the circumferential fibers, from the innermost to the outermost. While these assumptions may be applied to pipes having thin walls, they are greatly in error, when the pipe walls are thick.

Various formulas have been proposed for calculating the stresses in pipes or tubes having thick walls. Reid T. Stewart, M. Am. Soc. M. E., analyzed carefully the principal ones of these formulas, and tested their accuracy by experiments conducted at one of the mills of the National Tube Company.*

Mr. Stewart recommends the Barlow formula, given below, as best suited for all ordinary calculations, pertaining to the bursting strength of commercial thick tubes, pipes, and cylinders. The errors involved in its use are on the safe side, and it is much to be preferred to formula (70) for pipes having thick walls, for which this formula errs greatly on the side of danger.

The Barlow formula is as follows:

$$p = \frac{2ft}{D}, \tag{71}$$

where D = outside diameter in inches;
t = thickness of pipe wall in inches;
p = internal gauge pressure in pounds per square inch;
f = fiber stress in the pipe wall in pounds per square inch.

This formula is like equation (70) in form, the only difference being that the external diameter of the pipe is used instead of the inner diameter.

126. Water-hammer. — When the motion of water flowing in a pipe is suddenly checked or stopped, a force, called water-hammer or water-ram, is produced that is in excess of the static pressure in the pipe. In small pipes of a distribution system, this force may exert a pressure as great as 100 lbs. per square inch, and in larger pipes it may amount to 50 per cent of the static pressure. If the system of pipes is cross-connected, and the water is flowing at a velocity of only about 3 ft. per second, the effect of water-hammer is insignificant, but in a penstock in which water flows at a high velocity due provision must be made for water-hammer.

127. *Theoretical Formulas for Water-hammer.* — The greatest possible water-hammer will be caused by closing a stop-gate in a pipe-line practically instantaneously. Of course, this can only be done in a small pipe. In large mains the length of time required to close a gate prevents, or, at least, greatly reduces, the effect of water-hammer. When the velocity of flow is suddenly stopped, the resulting pressure is a function involving the elasticity of the water and of the pipe, and is a case of impact of an elastic prism. If we neglect the elasticity of the metal of the pipe, as may be done for ordinary sizes of pipe, the impact will be represented by the following formula:†

$$p = \frac{vE_w}{V}, \tag{72}$$

* Trans. Am. Soc. M. E., 1906, XXVII, p. 730.

† "Public Water-supplies," by F. E. Turneaure and H. L. Russell, 2d Edition, p. 243, New York.

where v = initial velocity of water;
E_w = modulus of elasticity of water
= 300,000 lbs. per square inch;
V = velocity of sound in water
= about 4700 ft. per second.

Substituting the values given above for E_w and V we obtain:

$$p = 64\,v, \tag{73}$$

where p = water-hammer in pounds per square inch and v = velocity in feet per second. From this formula we see that the pressure produced by water-hammer is proportional to the velocity of the water and independent of the length of the pipe.

J. B. Frizell, M. Am. Soc. C. E.,* obtained the following formula for water-hammer, taking the elasticity of the pipe into account:

$$p = \frac{v}{V} \times \frac{E_w}{1 + \frac{2\,r}{t} \times \frac{E_w}{E_p}}, \tag{74}$$

where E_p = modulus of elasticity of the pipe;
$2\,r$ = diameter of pipe in feet;
t = thickness of pipe in inches.

The theoretical formulas indicate the maximum possible pressure that might be produced by water-hammer. It is practically impossible to stop the velocity of the water instantly, except when the pipe is very small, and the pressure that may result from water-hammer must, therefore, be determined by experiment. Air that may collect in a water pipe has, also, an effect upon water-hammer.

Mr. Frizell considered the closing of a gate or valve to be essentially instantaneous, if the time of closing is less than the time required for the wave of pressure to be transmitted to the end of the pipe and back, at the rate of about 4700 ft. per second. This brings the length of the pipe into the problem. Shutting a pipe quickly, which would have no bad effect in a short pipe, might produce a dangerous water-hammer if the pipe were considerably longer.

Vincent P. Marran,† Superintendent of Walsh's Holyoke Steam Boiler Works, Holyoke, Mass., gives the following empirical formulas which he has used in designing long pipe-line:

$$P = 0.0201\,\frac{(L \times V)}{T}, \tag{75}$$

* Trans. Am. Soc. C. E., 1898, XXXIX, p. 1.
† *Eng. Record*, March 20, 1915, p. 355.

where P = pressure in pounds per square inch;
L = length of pipe-line in feet;
T = time in seconds, required to close gate.

If the closing of the gate in T seconds is greater than the time of a wave *round trip* from the gate to the reservoir and back, or T_s seconds, the intensity of the water-hammer will diminish as follows:

$$\frac{P}{P_t} = \frac{T_s}{T}. \tag{76}$$

The time T_s required for a wave of compression to travel twice the length of the pipe — that is, to make the round trip — is:

$$T_s = \frac{2L}{V_s}, \tag{77}$$

where L = length of pipe in feet;
V_s = 4700 ft. per second (approximately), which is the velocity of sound in water.

Mr. Marran states that the above formula has proved to be very satisfactory. He gives, also, the following formula as being frequently used:

$$P = \frac{LV}{gT}\left[\frac{1}{1 - \left(\frac{LV}{2\,gTh}\right)}\right], \tag{78}$$

where L = length of pipe in feet;
V = velocity of water in the pipe at the time the gate is closed;
g = 32.2;
T = time required to close gate in seconds;
h = net head in feet at time of gate closing;
a = velocity of wave propagation in feet;
P = excess pressure in feet of water-hammer.

This formula can be used when P is less than h and when $\frac{aV}{2\,gh}$ is greater than 1.

The velocity of wave propagation equals practically the rate at which flowing water is able to compensate for the compression of the water and the extension of the pipe, corresponding to the increased pressure. The proper value is the actual velocity of the wave, which is in many cases materially different from the given value of 4700 ft. per second. When the pipe extension is not considered, Prof. I. P. Church gives the formula:

$$\text{Velocity of wave} = \sqrt{\frac{Eg}{g}} = 4726 \text{ ft. per second}, \tag{79}$$

where E = bulk modulus of water = 300,000 lbs. per square inch;
g = 386.4 ins. per second = velocity due to gravity;
w = 0.03604 lbs. per cubic inch = weight of water.

For empirical formulas which do not include the pipe distension the value of 4700 ft. per second may generally be considered satisfactory.

128. *Experiments on water-hammer* have usually been limited to ascertaining the increase of pressure caused in short pipes by closing a stop-gate quickly. E. B. Weston determined by experiments made at Providence, R. I.,* on small pipes that the water-hammer was practically

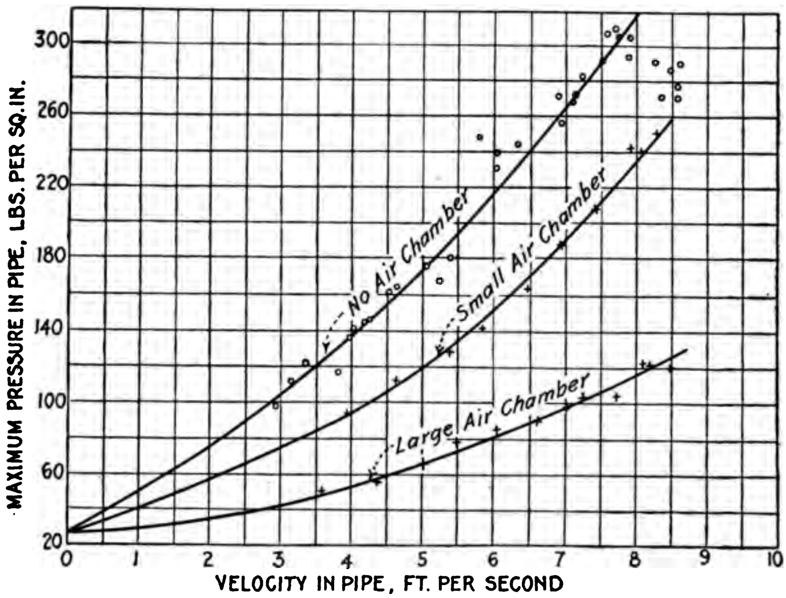


FIG. 76. — Experiments on Water-hammer.

proportional to the velocity of the water. The results he obtained approached closely to the maximum given by formula (72).

Prof. Carpenter† made experiments on 2-in. pipes, the results of which are shown in the diagram of Fig. 76. When no air-chamber was used, he found pressures from one-half to two-thirds of those obtained by using Frizell's formula (74). The pressure due to water-ram appeared in these experiments to increase more rapidly than the velocity. The effect of air-chambers in these experiments is very marked.

Experiments made at Dartmouth College‡ in 1898 showed that the

* Trans. Am. Soc. C. E., 1885, XIV, p. 238.

† *Ibid.*, 1894, XV, p. 510.

‡ *Eng. News*, Mar. 24, 1898, p. 186.

force of water-hammer varies with the velocity, and with the volume of water in the pipes. This force is increased when dead ends are near the gate. Extensive experiments made by N. Jonkowsky in Russia* confirm the general laws of formulas (72), (73), and (74). According to these experiments, the pressure caused by a sudden decrease in velocity is, for each foot per second of such decrease, approximately, 4 atmospheres (60 lbs. per square inch) for small pipes and 3 atmospheres (45 lbs. per square inch) for large pipes. These values approach very closely to those obtained by using formula (74).

The manner in which a gate is closed has an effect on the water-hammer. If the rate of closing is uniform, the maximum pressure will occur at the end of the movement, and with similar laws of closing the pressure will be approximately proportional to the length of the pipe, to the speed of closing the valve, and to the velocity of the flow. If the gate is closed rapidly at first, and more slowly toward the end of the movement, the water-hammer will produce a lower maximum pressure.†

129. Temperature Stresses. — The ordinary bell-and-spigot joint, filled with lead, permits cast iron pipes to expand or contract, when changes of temperature occur. In some of the early lines of riveted wrought iron or steel pipes, expansion joints were introduced, but in later works of this kind no allowance for expansion and contraction was made, unless the pipes were uncovered, as the metal of the pipe is generally strong enough to resist the stresses caused by changes of temperature.

The temperature stresses caused in a riveted pipe by changes of temperature may be calculated by the following formula:

$$S = cET, \tag{80}$$

where S = temperature stress in pounds per square inch;
E = modulus of elasticity — assumed at 30,000,000 lbs. per square inch for steel;
T = change of temperature in degrees Fahrenheit;
c = coefficient of expansion.

The coefficient of linear expansion of soft steel plates ranges from 0.0000062 to 0.0000068 for each degree Fahrenheit. Assuming 0.0000065 as a fair average value, and taking the modulus of elasticity

* "Memoirs of the Imperial Academy of Sciences of St. Petersburg," Vol. IX, No. 5. A synopsis of this paper is given in *Eng. News*, Aug. 2, 1900, p. 81, and in Trans. Am. Water Works Assn., 1904. See also Trans. Am. Soc. C. E., 1915, LXXIX, p. 281.

† Trans. Assn. C. E., Cornell University, 1898, p. 31. See, also, paper by Prof. I. P. Church in Journal of the Franklin Inst., April and May, 1890.

as 30,000,000, we find the stress caused in a rigid steel pipe by a change of temperature of one degree Fahrenheit to be 195 lbs. per square inch. For larger values of the coefficient of expansion and the modulus of elasticity, we would obtain greater stresses; but we may assume 200 lbs. per square inch as a fair average value for the stresses caused in a rigid steel pipe by expansion or contraction.

In a temperate climate, with the pipe covered by about $2\frac{1}{2}$ ft. of earth, the temperature of the steel may vary from 32 to 77° F. This would cause a stress of about 9000 lbs. per square inch in the steel, which would be in tension in winter in the parts that were laid in summer, and in compression in summer, in the parts that were laid in winter. In the intermediate portions, the stress might be only half of the maximum. If the pipe were laid at a temperature of 20° F. and raised to a temperature of 80 degrees in summer, the longitudinal compression would amount to about 12,000 lbs. per square inch. This stress would have to be transmitted by the circular joints, and might cause, if only one row of rivets were used in these joints, as is customary, a shearing strain as great as 15,000 lbs. per square inch on the rivets, which is nearly double the amount usually allowed in structural work.

130. Unbalanced Pressure in a Pipe-line. — At dead ends, closed gates, horizontal and vertical curves (bends) of a pipe-line, the water in the pipe produces an unbalanced pressure that causes longitudinal stresses in a pipe, unless it is anchored to its foundation. For ordinary water pressures these longitudinal stresses may be neglected, but for high pressures — such as exist for instance in high-pressure fire-lines — provision must be made to resist them. The pressures on dead ends and gates may be opposed by blocks of masonry to which the pipe is anchored, or by fastening the pipes together by tie rods, attached to lugs cast on the pipes, for a sufficient length to permit the friction between the pipe and its trench to neutralize the unbalanced pressure.*

The unbalanced pressure on a horizontal curve may be determined as follows:

In Fig. 77, let

P_1 and P_2 = the axial water pressures in the straight parts of the pipe-line, in lbs.;
R = resultant of P_1 and P_2;
d = diameter of pipe in inches;
p = water pressure, in lbs. per square inch;
a = central angle of the curve.

* The latter method was used in the *high-pressure fire system* of New York.

We shall have

$$P_1 = P_2 = \frac{\pi d^2}{4} p,$$

$$R = P_1 \sin\frac{a}{2} + P_2 \sin\frac{a}{2} = 2 P_1 \sin\frac{a}{2},$$

$$R = \frac{\pi d^2}{2} p \sin\frac{a}{2}. \qquad (81)$$

R may be increased or diminished by temperature stresses, computed by formula (80), according to whether expansion or contraction occurs.

R will be increased by the centrifugal force of the water, but this force is negligible, except where great velocities occur.

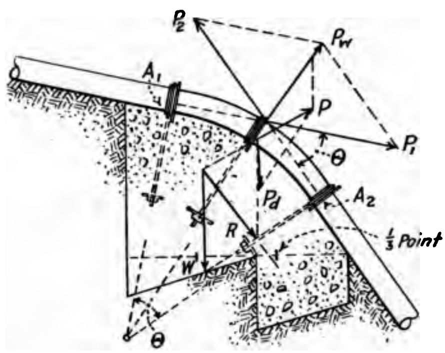


FIG. 77.

The centrifugal force C can be computed by the following formula:

$$C = \frac{WV^2}{2\,gR}, \qquad (82)$$

where W = weight of water in the bend in pounds per square inch;
V = velocity in feet per second;
R = radius of curvature of the bend in feet.

In the case of a vertical concave curve, the unbalanced water pressure and the temperature stresses are neutralized by the reaction of the foundation of the pipe trench. If the curve is, however, convex, the unbalanced pressure may necessitate the anchoring of the pipe to a block of masonry or to a rock foundation. In this case the unbalanced pressure R is partly neutralized by the weights of the pipe, the water it contains, and of the refilling. Calling the resultant of these weights W and combining it by the parallelogram of forces with R (Fig. 77) we obtain R as the final resultant acting on the bend.

If the pipe-line is laid on a steep grade, the component of the force of gravity, parallel with the axis of the pipe-line, will cause the up-stream part of the pipe-line to push, and the down-stream side to pull, on the bend. These forces will produce additional longitudinal stresses on the bend, unless they are neutralized, as is often done, by the construction of anchor piers to which the pipe-line is tied.

131. Beam-action in Pipes. — Where a pipe is supported by piers, and is not covered, it forms a continuous beam. The stresses that will be caused in the shell of the pipe by the weights of the pipe and of the water can be determined by mathematical formulas.* When the pipe is covered by refilling, which is well tamped around the pipe, beam action will not take place, but the refilling will tend to collapse the pipe, especially when it is empty. Under ordinary circumstances cast iron or wooden pipes are strong enough to bear any weight of filling which is likely to be placed upon them, but thin wrought iron or steel pipes may be distorted by the earth filling. In estimating the crushing pressure to which a pipe may be exposed, the possibility of vacuum in the pipe, more or less complete, caused by a break letting the water out quickly, must be considered.

The collapsing pressure exerted by the refilling on the empty pipe cannot be computed accurately, as it depends upon a great many conditions, such as the kind of material used for refilling, the care with which it was deposited and rammed, etc. When the covering on the pipe is only 2–3 ft. deep, the pressure produced by its weight on the pipe may be neglected. For greater depths of filling, the stresses produced by the backfilling can be computed approximately in the following manner:†

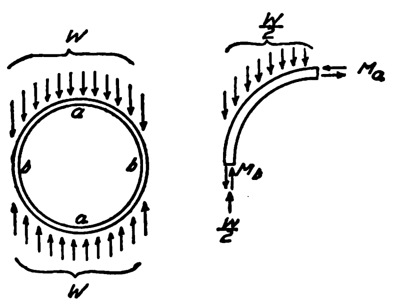


FIG. 78.

In Fig. 78, the weight of the filling on top of the pipe is assumed to be distributed uniformly over a width equal to the diameter of the pipe, as a vertical load, and the lateral support given by the filling is neglected. The weight of the filling is neutralized by the reaction of the foundation, acting upwards. At the points a–a and b–b there will be produced equal bending moments, but of opposite signs, because a similar set of horizontally applied forces must reduce the moments at a and b to

* Hütte, "Des Ingenieurs Taschenbuch," Band 1, "Durchlaufende Träger."

† "Public Water Works," by F. E. Turneaure and H. L. Russell, 2d edition, p. 499, New York.

zero.* Representing the total load in pounds by W and the diameter of the pipe in inches by d, we find the bending moment at the points a and b to be:

$$M = \tfrac{1}{16} Wd. \tag{83}$$

Assuming h = depth of fill in feet; weight of filling = 100 lbs. per cubic foot; f = safe fiber-stress in bending for the pipe material; d = diameter of pipe in inches; t = thickness of pipe in inches, we derive, from the ordinary beam formula approximately:

$$t = \tfrac{1}{2} d \sqrt{\frac{h}{f}}. \tag{84}$$

132. Collapse of Riveted Pipes. — When the pipe is full of water, the hydrostatic pressure on the inside will generally prevent the pipe from collapsing, and will maintain its circular form. The danger of a thin pipe's collapsing † will occur when the pipe is empty and the back filling is heavy; or when a break in the pipe-line, or the sudden opening of a gate, permits the water to rush out of the pipe with such a velocity that the hydrostatic pressure becomes practically zero, or a minus pressure, causing a partial vacuum. Air-valves are usually placed at different points on a line of riveted steel pipes, to permit the air in the pipe to escape during the filling, air that may be contained in the water to leave the pipe at summits, and to admit air into the pipe, as the water is drawn off. In most cases the air-valves are entirely too small to be effective in preventing the formation of a partial or a complete vacuum, in case of a break. In some pipe-lines the air would have to flow through the air-valves with a velocity of 1000 to 2000 feet a second to enter the pipe as rapidly as the water would flow out, in case of a serious break.

M. L. Enger and F. B. Seely have given the following formula,‡ based upon a theoretical investigation, for the excess of external over internal pressure, which a riveted pipe can withstand without collapsing.

$$p_2 - p_1 = 77{,}300{,}000 \left(\frac{t}{d}\right)^3, \tag{85}$$

where p_2 = external pressure in pounds per square inch;
p_1 = internal pressure in pounds per square inch;
t = thickness of shell of pipe in inches;
d = diameter in pipe in inches.

* Paper by Wm. H. Searles on "Deflections and Strains on a Flexible Ring under Load," Jour. Assoc. Engineering Societies, 1895, XV, p. 124.

† By the collapsing of a pipe the buckling inward of the plates is meant. This will not always rupture the pipe, which may resume its original circular form, as soon as the pressure inside of the pipe exceeds that on the outside.

‡ *Eng. Record*, May 23, 1914, p. 594.

If the pipe is not perfectly round, and the refilling is not homogeneous, it will take less pressure to collapse the pipes.

Thus far, no experiments have been made on the resistance of large riveted pipes to collapsing, but they have been made on boiler tubes and well-casings.* According to these experiments, small cold-drawn and lap-welded tubes, less than 10 ins. in diameter, will collapse, when the excess of inward over outward pressure reaches that given by the following formula, in which the letters have the same significations as in formula (85):

$$p_2 - p_1 = 50{,}200{,}000 \left(\frac{t}{d}\right)^3. \tag{86}$$

This empirical formula may reasonably be expected to apply to similar pipes of larger diameter having smaller values of $\frac{t}{d}$ than those experimented upon. In the case of a riveted pipe with lap-joints, the effect of the longitudinal joints will be to cause the pipe to collapse with a lower pressure than that given by the formula (86), while the circular joints will increase the strength of the pipe, although not as much as might be thought, this increase being probably less than 4 per cent.

Assuming the riveted pipe to be as strong as indicated by formula (86), Enger and Seely have computed the following table to show the pressures at which pipes would collapse.

TABLE XXXIX. — PRESSURES AT WHICH RIVETED PIPES WILL COLLAPSE

$p_2 - p_1$ lbs. per square inch	$\frac{t}{d}$	Least diameter of pipe which will collapse		
		$t = \frac{1}{4}$ in.	$t = \frac{5}{16}$ in.	$t = \frac{3}{8}$ in.
14.7	0.0066	38.0	47.5	57.0
10.0	0.0059	47.5	53.0	64.0
9.0	0.0056	44.5	56.0	67.5
8.0	0.0054	46.5	58.0	70.0
7.0	0.0051	48.5	61.3	73.0
6.0	0.0049	51.5	64.0	77.0
5.0	0.0046	54.5	68.0	82.0
4.0	0.0043	59.0	73.0	88.0
3.0	0.0039	64.5	80.0	97.0
2.0	0.0034	74.0	92.0	111.0
1.0	0.0027	93.0	116.0	139.0
0.5	0.0021	117.0	149.0	176.0

* Experiments by Prof. R. T. Stewart on "Collapsing Pressures of Bessemer Lap-welded Tubes, 3-10 ins. in Diameter," Trans. Am. Soc. M. E., 1906, XXVII, p. 730. See, also, experiments by Prof. A. P. Carman and M. L. Carr in 1906 on "Resistance of Tubes to Collapse," Bulletin No. 5 of the University of Illinois Experiment Station. The experiments of Carman and Carr supplement and corroborate those of Prof. Stewart.

From the above table it appears that if the riveted pipes with lap-joints were relatively as strong as lap-welded pipe — which they, probably, are not — they would collapse under a relatively low partial vacuum. This shows the importance — especially for riveted pipes laid on steep grades — of either giving the pipe sufficient strength to be able to carry the external load with a complete vacuum in the pipe, or of providing air-valves large enough to admit air as rapidly as the water can run out, in case of a complete break. The former plan was adopted in the water power works on the Loutsch in Switzerland.*

Several riveted steel pipes have failed by collapsing. Parts of the Bull Run pipe-line, which conveys the water supply to Portland, Ore., collapsed in 1911.† When the pipe was tested, a rupture occurred at a low point, which caused twenty-two sections of lock-bar pipe, 52 ins. in diameter and $\frac{1}{4}$ and $\frac{5}{16}$ in. thick, to collapse. Another part of the pipe-line of less extent collapsed for the same reason. A third collapse occurred while the pipe was being drained, after a test. The air-valves provided on the pipe-line were much too small to admit air rapidly enough to prevent the formation of a partial vacuum. Portions of the pipe-line, which had been covered with earth, did not collapse, which shows that the refilling gives a pipe under such circumstances additional strength.

Part of the steel pipe of the Antelope siphon of the Los Angeles aqueduct (p. 319) collapsed in 1914.‡ This siphon, which is 10 ft. in diameter and 21,747 ft. long, is subjected to a maximum head of 200 ft. For a distance of about 2700 ft. on the north end and for 3400 ft. on the south end the siphon is constructed of reinforced concrete. For the remaining distance steel pipe made of plates, $\frac{1}{4}$ to $\frac{3}{8}$ in. thick, are used, the greater part of the longitudinal joints being triple riveted. The pipe is supported on masonry piers.

The failure occurred after a severe rain storm, lasting three days, which increased the volume of the run-off at the lowest point of the siphon to such an extent, that it undermined two of the masonry piers and turned them over on their sides. The pipe being thus dropped at the lowest point of the siphon, three of the circular seams were torn apart and let the water escape with great velocity. Although both ends of the pipe were open to the atmosphere, the rapid rush of the water required most, if not all, of the static head in the pipe to be converted into velocity head. There was, therefore, practically no pressure on the inside of the pipe to oppose the atmospheric pressure on the outside, and,

* *Schweizerische Bauzeitung*, April 16, 1910.

† *Eng. News*, July 27, 1911, p. 112.

‡ *Eng. Record*, April 18, 1914, p. 447.

with the exception of a short stretch, all of the pipe which had a thickness of shell of only $\frac{1}{4}$ or $\frac{5}{16}$ in. collapsed. The pipe near the bottom of the siphon, which was $\frac{3}{8}$ in. thick, remained intact except at the points of rupture. When the damaged parts had been repaired and the water was turned on again, the collapsed sections of the pipes resumed their former circular form.

CHAPTER IX

FLEXIBLE PIPE-JOINTS

133. Flexible joints are used in pipe-lines which are laid under water on the bed of a lake or river, and in other places where settling is likely to occur. Quite a number of joints of this type have been designed and used successfully, especially in submerged pipe-lines. In some cases, every joint of the pipe-line is made flexible; in others, flexible joints are placed in the pipe-line at certain intervals.

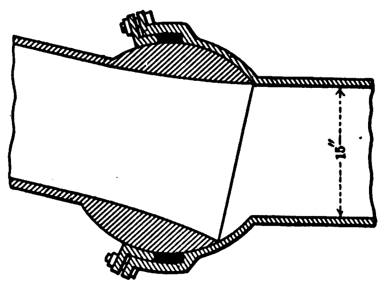


Fig. 79. — Flexible Pipe-joint, Invented by James Watt.

The first flexible joint (Fig. 79) was designed by James Watt upon the *ball-and-socket* principle. The various modifications which have since been introduced in flexible joints are described in the following pages.

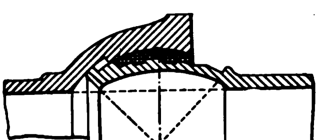


Fig. 80. — Turned Hub.

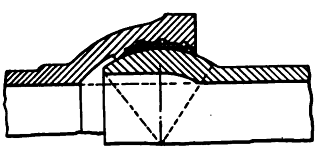


Fig. 81. — Turned Spigot.

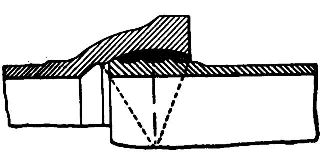


Fig. 82. — Stop Rings on Spigot.
Ward Flexible Joints.

134. American Flexible Pipe-joints. — In 1862 John F. Ward, who was at that time superintendent of the Warren Foundry and Machine Co., of Philipsburg, New Jersey, invented a flexible pipe-joint, which was used in a line of 8-in. pipes, laid across the Delaware River from Easton, Pennsylvania, to Philipsburg, New Jersey.* In the following year Ward took out a United States patent for this device, which he called "An Improvement in Pipe Coupling." This invention, known as the *Ward joint*, has been extensively used in the United States for pipes 6 to 36 ins. in diameter. It is made in several ways (Figs. 80, 81, and 82). In Fig. 80 the bell is bored to a true spherical surface for the required overlap on

* "Engineering and Contracting," April 15, 1914, p. 432.

each side of the central diameter, and the cylindrical spigot is thickened and grooved, the end being turned in a lathe. The annular space between the bell and spigot is filled with melted lead, which is held to the spigot by one or more grooves and slides over the smooth spherical surface of the bell, when the spigot is deflected in any direction. If any leakage occurs, the lead at the face of the bell can be caulked. The retreat of the edge of the lead within the bell on one side, and its projection beyond the bell on the opposite side, makes caulking difficult. This may be overcome by giving the spigot a smoothly turned spherical surface, while the interior of the bell is cast with grooves to hold the lead immovably, and a projecting ring at the inner end of the lead is bored to match the smooth surface of the

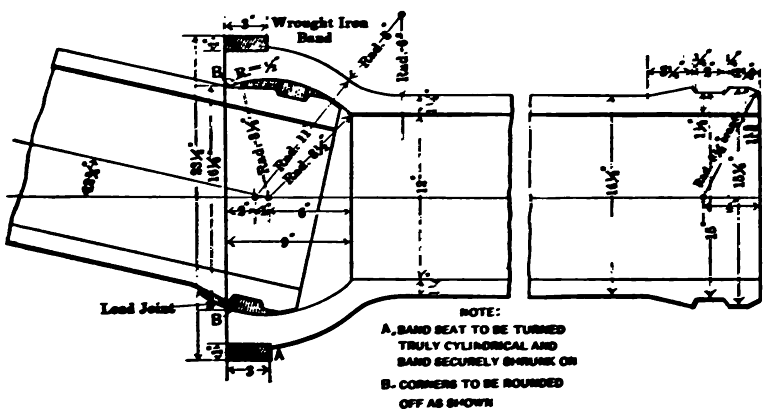


FIG. 83. — Duane Pipe-joint.

spigot, as shown in Fig. 81. In this style of joint, the outer edge of the packing is always flush with the face of the bell, and can, therefore, be readily caulked. Another modification of the Ward joint is shown in Fig. 82. This joint is designed for cases in which it is desirable to limit the degree of flexure and to bring the parts to a firm bearing, without injury to the lead packing, or danger of splitting off the inner edge of the bell. This is accomplished by providing two stop rings on the spigot, and a shoulder in the bell.

135. *Duane Joint.* — The Ward joint was used extensively, but occassionally the bell was found to break, when the joint was subjected to a strong longitudinal pull. To overcome this difficulty, James C. Duane, M. Am. Soc. C. E., who laid a number of submerged pipes for the City of New York, devised an improvement in the Ward joint, which consists in reinforcing the bell by shrinking a wrought iron or steel band around it.* Another improvement devised by Mr. Duane is to round off the inner edge of the bell, in order to prevent it from cutting the lead when the joint is moved. *The Duane joint* (Fig. 83), as it is called, has been used very successfully for submerged pipes, 6 to 36 ins. in diameter.

136. *Fanning Joint.* — In 1875 J. T. Fanning, M. Am. Soc. C. E., invented a flexible joint with lead packing (Fig. 84) for a 24-in. submerged pipe.† The difficulty of making the back part of the lead-packing of the joint firm and solid, which had interfered with the complete success of large flexible pipes, is overcome in this joint by separating the bell into two parts, so as to permit both the front and rear parts of the packing to be caulked. In putting this joint together, the loose ring is passed over the ball-spigot, and slipped some distance toward the center of the pipe; the ball-socket is then entered into the solid part of the bell, and its lead-joint-packing poured and snugly caulked. The loose ring is then bolted in position, and its lead-joint-packing is poured and caulked. This secures a solid packing at both front and rear of the joint, capable of withstanding the strain that comes upon it as the pipe is lowered into position, and insures a tight joint. The ball-spigot is turned smooth in a lathe to true spherical form.

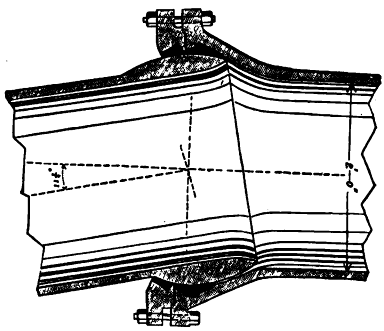


FIG. 84. — Fanning Pipe-joint.

137. *Smith Joint.* — In 1893 I. W. Smith, Chief Engineer of the water works of Portland, Ore., modified the Fanning joint, and used the improved joint for a submerged line of 28-in. cast iron pipes, laid across the Willamette River (p. 169). The pipes were cast in the unusual length of 17 ft., but, when united, the length from center to center of ball-joint was only 16 ft. The large overlap of $6\frac{1}{4}$ ins. on the ball, the exterior diameter of which was 40 ins., caused the smallest diameter of the clamp-ring to become about 39 ins., which is less than the diameter of the ball. This necessitated making the clamp-ring in two halves with flanges for bolting together; and in order to transmit the bursting force, developed by a pull of the joint, to the continuous flange of the bell, the split clamp-ring was recessed into the bell-flange as shown in Fig. 85. The bell was strengthened longitudinally by thirty-two equidistant ribs. These joints permitted a deflection of 15 degrees.

* *Eng. News*, May 18, 1889, p. 444.

† Fanning's "Hydraulic and Water Supply Engineering," 9th Edition, p. 463.

In 1897 a line of lap-welded steel pipes, having an exterior diameter of 24 ins., was laid across the Willamette River at Portland, Ore. (p. 171). The pipes, which are 18 to 20 ft. long, were connected by means of cast iron flanges, and the flexible joint (Fig. 86) * was inserted in every second length.

138. *Holmes Joint.*† — A. L. Holmes of Grand Rapids, Mich., invented the joint shown in Fig. 87. It consists of a ball, two bells and

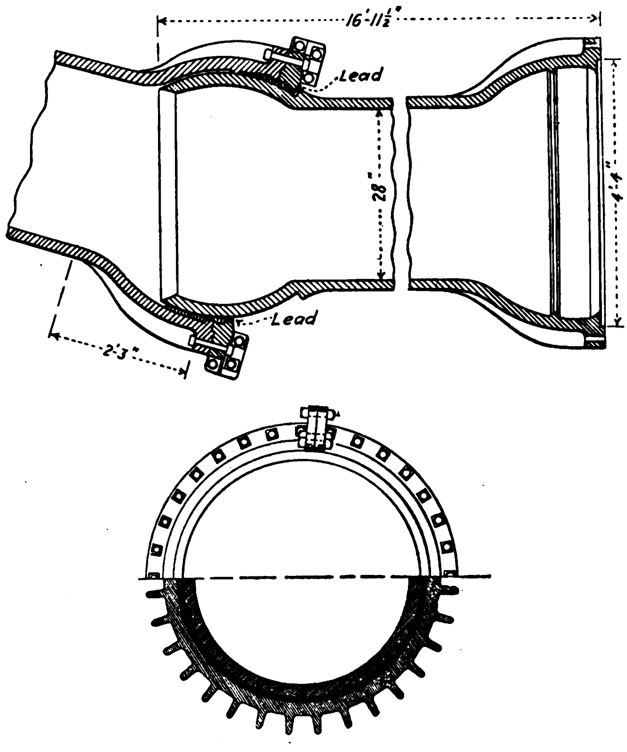


Fig. 85. — Submerged 28-in. Cast Iron Pipe, Portland, Ore.

two clamp-rings with their necessary bolts. The spherical surfaces are carefully turned in a lathe. A projecting copper ring, placed in a groove near each end of the ball, insures water-tightness.

A slight modification of the Holmes joint was made in the steel high-pressure water mains of Baltimore, Md. (Fig. 88). These pipes are 8 to 20 ins. in diameter, and 20 ft. long. Fig. 88 ‡ shows the joint used

* *Eng. News*, Feb. 10, 1898, p. 100.
† *Ibid.*, Nov. 22, 1890. p
‡ *Eng. Record*

for the 8-in. pipe. All spherical surfaces are turned accurately in a lathe. While a joint is being made, the steel segmental ball is held erect by means of a ring-bolt, screwed into threaded hole at the top, until its edges are parallel with the two steel clamp-rings. The clamp-bolts are then drawn up, to bring the bell in contact with the ball.

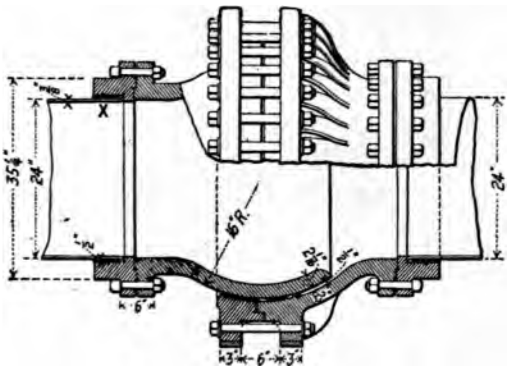


FIG. 86. — Smith Pipe-joint.*

139. *Law Joint.* — Riveted steel pipes, 48 to 60 inches in diameter, were used in the long intake-pipe of the Toronto Water Works (p. 223), in 1891. The flexible joints (Fig. 89) placed in this pipe-line were designed by Wm. H. Law, C. E., of the Central Bridge and Engineering Co., Peterboro, Ont., and in 1893 a number of similar joints were designed by him for a line of 72-in. steel pipe, laid in the same city. The

FIG. 87. — Holmes Pipe-joint.

joint devised by Mr. Law has since been adopted, with slight modifications, by other engineers for large steel intake-pipe.

140. *Stucker Joint.* — In 1895 N. E. Stucker designed a flexible joint (Fig. 90) for a line of 14-in. cast iron pipe, 550 ft. long, which was laid across the Illinois River at Ottawa, Ill.† This joint is only flexible, while being adjusted, and then remains rigid. The bolts are of different lengths, and beveled washers are needed, to provide a proper bearing for

* *Eng. News*, Feb. 10, 1898, p. 100.

† *Ibid.*, April 18, 1895, p. 254.

head and nut, unless they are suitably bent. Water-tightness is secured, in some cases, by inserting and compressing a narrow ring of lead or copper in a groove, cut in each bell, at the bottom of the flange. In the pipe laid at Ottawa wedged-shaped wooden gaskets were placed

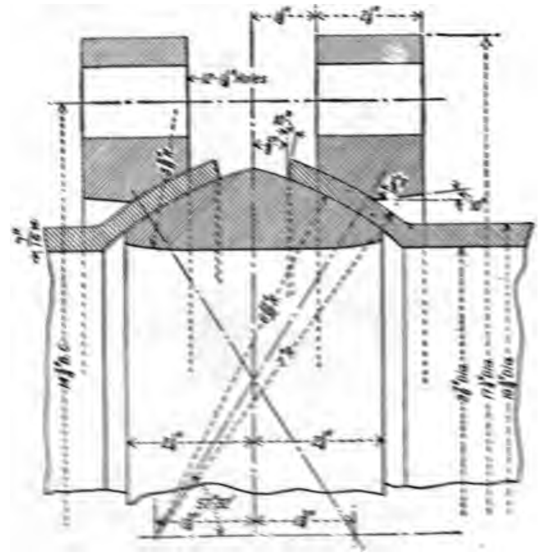


FIG. 88. — Holmes Pipe-joint, Used in Baltimore, Md.

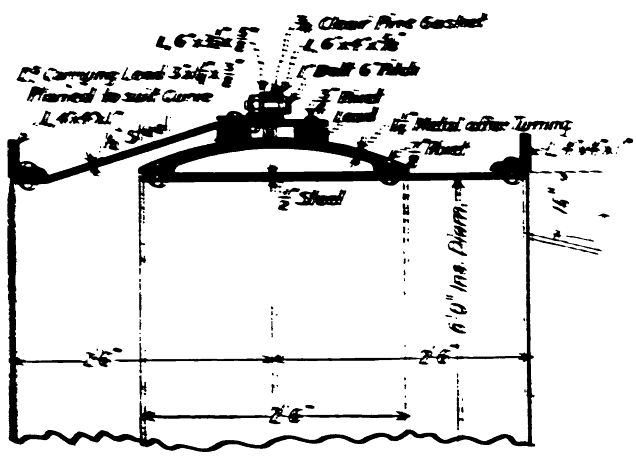


FIG. 89. — Law Flexible Pipe-joint.

between the flanges of the ball-joint, and plain sheet-rubber gaskets at the end flanges. The ball was kept from turning independently of the bells by a hinged bolt on each side. All bolts were inserted and drawn up by divers.

141. *Various Flexible Joints.* — Joseph G. Falcon, of Evanston, Ill., invented a flexible joint (Fig. 91)* for pipes which has been successfully used for submerged pipes up to 24 ins. in diameter.

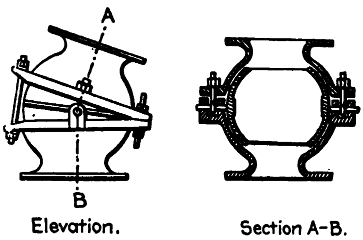


FIG. 90. — Stucker Pipe-joint.

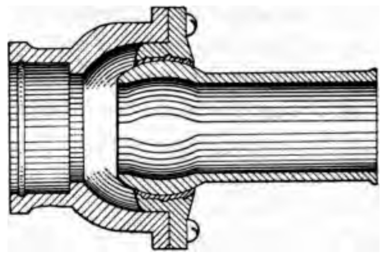


FIG. 91. — Falcon Pipe-joint.

Coldwell-Wilcox Co., of Newburgh, N. Y., make a flexible joint (Fig. 92) which can be used both for steam and water pipes. In this joint, the clamp-ring is fastened to the bell of the pipe by means of tap-bolts.

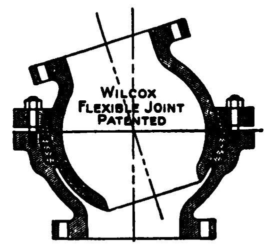


FIG. 92. — Wilcox Pipe-joint.

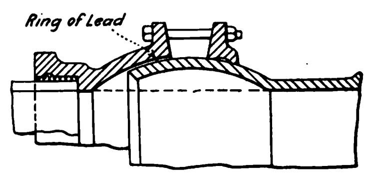


FIG. 93. — Walker Pipe-joint.

The Walker Manufacturing Co. of Fenton, Mich., makes flexible joints (Fig. 93) for pipes up to 24 ins. in diameter. Water-tightness is sought at the bell, by the compression of an inlaid hoop or ring of lead, while at the clamp-ring only a suitable bearing is required.

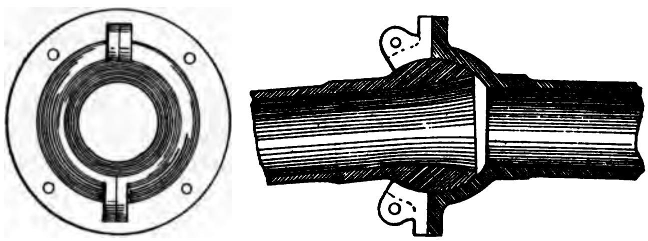


FIG. 94. — Humber Pipe-joint.

142. European Flexible Pipe-joints. — In submerged pipes, laid in various parts of Europe, subsequently to the siphons put across the River Clyde at Glasgow, modifications were introduced in the ball-and-

* *Eng. News*, June 8, 1893, p. 532.

socket joints. Fig. 94 shows a ball-and-socket pipe, given in Humber's "Treatise on the Water Supply of Cities and Towns," London, 1876. In this design, the clamp-ring, which is bolted to the bell, is made in two halves, in order to transmit the tendency to break it by a pull on the joint to the flange of the bell. Water-tightness is ultimately obtained by rusting, or the accumulation of fine silt.

143. Fig. 95 gives an early French design, known as the *Badois joint.** The special feature of this design consists in cutting away the heels of the diametral flanges to form a triangular groove, into which a thick cord of India rubber is placed, and powerfully compressed by the flange bolts, thus securing water-tightness. The clamp-ring is continuous, and the diameter of the ball is made large enough to allow the ring to pass over the end flanges. The Badois joint is shown in nearly all French treatises on water supply, published since 1860. The joint is

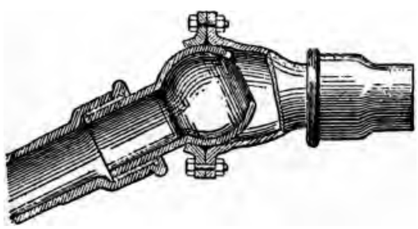


FIG. 95. — Badois Pipe-joint.

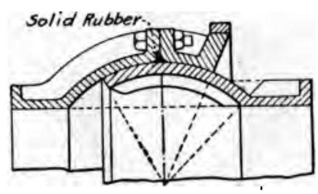


FIG. 96. — Jongh Pipe-joint.

also made with ordinary spigot ends for use with connecting pipes, having the common sockets.

144. Fig. 96 shows a ball-and-socket joint designed in 1892 by *G. J. De Jongh, C. E.*, for a 24-in. submerged pipe at Rotterdam, Holland.† It is characterized by a large overlap of the clamp-ring, the reinforcement of the latter with a thick steel ring; shrunk upon the radial flange, and the strengthening of the ball, bell and flanges by means of ribs. The dimensions are such that the clamp-ring can pass over the end flanges and allow a deflection of 26 degrees. Water-tightness is obtained by means of a ring of solid rubber, which is compressed in the triangular groove, formed at the heel of the diametral flanges. These joints weigh 5075 lbs. each.

145. Fig. 97 shows a joint having a moderate flexibility that was devised by an English engineer, *H. J. Marten.* It was used for a 22-in. main, which settled without leakage a vertical distance of 13 ft. in a

* "Distributions D'Eau," par Georges Dariès, Paris, 1909, p. 163.

† Trans. Dutch Society of Engineers, 1893.

length of 300 to 450 ft., owing to a subsidence of the ground, caused by mining operations.*

146. Fig. 98 gives a flexible joint, designed by *W. Williams* of Liverpool.† The spigot of the pipe is provided with a short spherical surface, the diameter of which is slightly greater than that of the cylin-

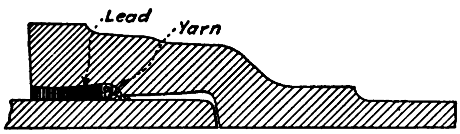


Fig. 97. — Marten Pipe-joint.

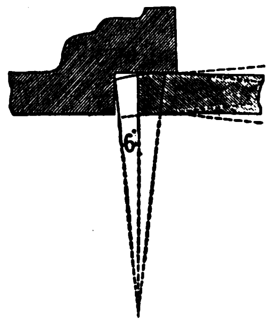


Fig. 98. — Williams Pipe-joint.

drical socket, both surfaces being carefully turned and bored. The socket is strongly reinforced by means of a ring of wrought iron, which is shrunk upon it. Before the spigot end is inserted, the bell is expanded by heating. Upon cooling, it grips the spigot powerfully. Although this joint is said to give excellent results, it has not been used extensively.

* Proc. Inst. C. E., Vol. LXXIV, p. 176.

† *Engineering*, London, July 9, 1869, p. 34.

CHAPTER X

SUBMERGED PIPES

147. Submerged pipe-lines have often to be laid across water courses, or to be extended into a river or lake, to form the intake of a water supply. When the pipes are covered by water, they are said to be submerged. Sometimes such pipes can be carried across a water course on an existing bridge; or on a light pile bridge, built for this purpose. Occasionally, the pipes have been laid in tunnels, driven under the river. The Mersey tunnel, constructed for a pipe-siphon of the Vyrnwy aqueduct of Liverpool, is an example of this kind. The more usual method of making the crossing of a water course is, however, to lay the pipes in a trench, dredged in the river-bed. If the water is shallow, the pipes may be laid in coffer-dams, as on land. When this method is impracticable, the pipes may be provided with flexible joints, and lowered into the water from a scow; or they may be floated in sections of several pipes to the desired position, and sunk into place, the joints at the junctions of the different sections being made by divers. In some cases, the pipe-line has been put together in sections on shore, and the sections have then been dragged by a steam engine into the dredged trench in the river.

148. Submerged Pipes in Glasgow, Scotland. — The first pipe-line with flexible joints was laid in 1810 across the River Clyde for the Glasgow Water Works. The pipes were connected by ball-and-socket joints (Fig. 79), invented by James Watt, whose connection with the work is described as follows:*

"Watt was consulted about the serious problem of laying a large water pipe across the river, and his solution was soon ready. A few days previously, he had seen a lobster on the table, and had discovered how the mechanism of the crustacean's tail might be reproduced in iron, so as to form a series of articulations, which would have complete flexibility. He suggested, therefore, the use of an articulated iron pipe, which would be able to adapt itself to all present and future irregularities in the bed of the river. The Glasgow Water Co. thereupon ordered

* Arago's "Éloge Historique de Watt," read before the French Academy of Sciences on December 8, 1834, quoted in *Engineering and Contracting*, April 15, 1914, p. 434. The flexible joint invented by Watt is also mentioned in Burnett's "History of the Water Supply of Glasgow," published in 1869.

such a pipe, 1000 ft. long, to be made and laid in accordance with Watt's plans, and its success was complete.

The cast iron pipe,* which was 15 ins. in diameter, was made in 9-ft. lengths, with flanged ends. The ball-and-socket joints were interposed at variable distances, depending upon the profile of the bottom of the trench. These joints were generally placed 28.5 ft. apart, except at the middle of the line, where the distance between the joints was reduced to 10 ft., in order to fit the ground. The ball surfaces were not turned, water-tightness being obtained by means of an adjustable packing-joint, similar to an ordinary stuffing-box.

The pipe-line with its flexible joints was put together on the south bank of the river on strong frames, made of parallel logs. These frames were joined by strong iron hinges, having their pivots in horizontal lines, at right angles to the axes of the pipes, and passing through the centers of the spheres of which the zones of the sockets formed portions. The flexible joints were at the extremities of the frames. After the frames and pipes had been put together, in succession, on ways, and the north end of the pipe had been plugged, the whole pipe-line with the frames was hauled into and across the river-bed in a trench, by machinery placed on the north bank of the river. The operation was quite simple, and was assisted by the buoyancy of the empty pipe and by pontoons. The movable joints and the hinges of the frames allowed the pipe line to conform to the grade of the pipe trench.

Three more similar lines of submerged pipes were laid across the River Clyde for the same water company, *viz.*: A line of 18-in. pipe, in 1830; a line of 25-in. pipe in 1838; and a line of 40-in. pipe in 1840.†

149. Submerged Pipe at Liverpool, England. — In 1891 George F. Deacon laid a line of 12-in. lap-welded steel pipe, 800 ft. long, on the irregular bed of the Mersey River, near Liverpool, in order to furnish a temporary water supply, during the construction of the Mersey tunnel.‡ He used in this line spherical cast-iron joints with lead packing (Fig. 99). The exterior of the ball and the rear interior portion of the bell were turned to a true spherical surface, and the front portion of the bell was provided with two $\frac{3}{4}$-in. holes, 1 in. apart, through which melted lead was poured, and, also, with a $\frac{3}{8}$-in. vent hole, which permitted the gases to escape during the operation.

The bed of the river, at the point selected for the crossing, is very shifting, and different banks of silt form at nearly every tide. As the current in the river is from 3 to 4 ft. a second at neap tide, great care

* Trans. Am. Soc. C. E., 1895, XXXIII, p. 284.

† See "Life of James Watt," by M. Arago, 1833.

‡ *The Engineer*, London, June 5, 1891, p. 442.

had to be taken to keep the pipe from being moved laterally. The whole line, consisting of 50 sections of pipe, was put together in a shore trench on wooden rails or ways, resting on the ground. Blocks of wood were placed under the pipe at each section joint so as to raise the pipe and joint from the ways. These blocks were fitted to slide along the ways, which were lubricated with soft soap. A wire rope was placed at each side of the pipe-line and was passed through the wooden blocks. These ropes were secured at the extreme end of the pipe from the river, and extended across the river to the other bank, where they were fastened to a steam winch, which pulled the whole pipe, with the blocks attached to it, across the river, in one operation. For about 200 ft. from the river, the trench had a considerable gradient. Eight boats, moored across the river at regular distances, served to guide the pipe. Two boats kept at the head of the pipe, and held the wire cable, keeping

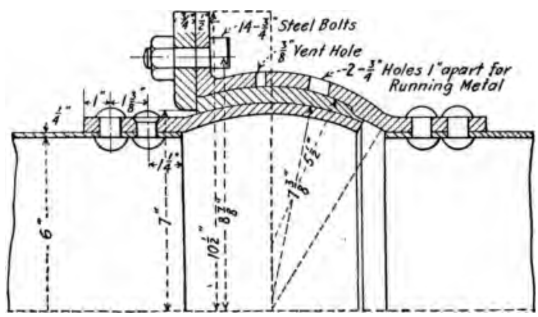


FIG. 99. — Liverpool Pipe-joint.

thus the head of the pipe always up. The tendency of the ebb tide to wash the pipe down, before being sunk, was counteracted by guy ropes from a heavy rope, stretched across the river, and carried over the sterns of the large boats, moored in the river fore and aft. The guy ropes were connected with timber bars, lashed across the pipes, to prevent the pipes from overturning in the water. The pipes were laid slightly zig-zag in the cradle, and were drawn across the river in this position, in order to allow for expansion of length, according to what the pipes might require after being sunk.

By means of the steam winch on the far shore, and of horses, the whole line of pipe, 800 feet long, was pulled across the river in 27½ minutes. The pipe worked itself gradually into its proper position, and, upon being tested, all pipe-sections were found to be sound.

150. Submerged Pipes in New York. — A number of institutions were built by the City of New York on Islands in the Harlem and East Rivers. In order to supply these buildings with water, pipe-lines, 1000 to 1300 ft. long, had to be laid to them in water 30 to 100 ft. deep. Randall's, Ward's, and Blackwell's Islands were supplied in this manner in 1850. For the first two of these islands, lead pipes, 3 ins. in diameter, laid in wooden trunks below the bottom of the East River, were used with perfect success. At Blackwell's Island other means had to be employed, owing to the very irregular and rocky nature of the river bottom at this place. At first the island was supplied by means of gutta-percha pipes, but these were soon abraded on the rocks by the swift and frequently changing current of the tides. Occasionally, the pipes were torn by vessels dragging their anchors.

In 1871 the gutta-percha pipes were replaced by 3-in. wrought iron pipes, but they were not found to be strong enough to resist the dragging of the anchors by large vessels. Another difficulty experienced was the freezing of the water in the pipes, that were laid in salt water which remains fluid at a lower temperature than fresh water.

In 1873 a heavy 6-in. wrought iron lap-welded pipe was laid to Blackwell's Island at the deepest and widest part of the river, where the bottom was comparatively regular. The pipes were fastened together with heavy screw couplings, which were strengthened and protected by strong cast-iron sleeves, having lead joints. In this manner the joints were made as rigid as the other parts of the pipe-line. The pipe was placed in a heavy oak case, which was saturated with coal-tar, securely bolted and riveted together, the space between the pipe and the case being filled with hydraulic cement. The pipe-line, as fitted in the box, was 1350 ft. long, and weighed over 200 tons. The whole length of pipe was pulled across the river by a powerful dredge having a steam-capstan of over 100 horsepower, which hauled on a heavy chain-cable, fastened to the rocks of Blackwell's Island. Three powerful tug-boats assisted the dredge, and would have been able to hold up the end of the pipe, if the chain had broken. At the place where the crossing was made, the river has a maximum depth of about 100 ft. Similar pipe-lines were laid in 1874–84 to other islands in the East and Harlem Rivers. Wrought iron pipe-lines were found, however, to be very expensive in first cost, and difficult to maintain and repair. For these reasons, cast iron pipes with flexible joints were used after 1887 for submerged pipes within the limits of the City of New York. At first the *Ward joint* (p. 146) was adopted for these pipe-lines. In 1888 a line of 6-in. cast iron pipes, provided with these joints, was laid to Blackwell's Island.* It was about 950 ft. long, and cost, including water meters, etc., $8410, or about $8.90 per ft. The pipe was laid in the following manner: A heavy chain was stretched from shore to shore, and securely fastened to piles, driven for

* Proc. Municipal Engineers, City of New York, 1904, p. 133.

this purpose. It acted as a guide for the pipe laying. The pipes were laid by a large floating derrick, which picked up the chain and laid it across the deck. Heavy anchors were placed at the bow and stern, to assist in keeping the derrick in line. The pipes were laid like a chain, of which they formed the links. Each joint was poured and caulked, before the pipe was lowered into the water. The floating derrick was moved by hauling the guide chain slowly across the deck, the anchors being reset by a tug, where required. The joints used were capable of being rotated through 10 to 15 degrees of an arc. During the process of laying, several lengths of pipes were unsupported from the river-bed to the deck of the scow. The weight of these pipes assisted in compressing the lead joints and making them water-tight. When this pipe-line was tested under hydraulic pressure, it was discovered that the tops of several of the hubs of the flexible joints had been broken by the strain to which they had been subjected. The joints were repaired by means of clamps and sheet lead.

In order to remedy the defects of the flexible joints used in the submerged pipe described above, James Duane devised the improvements in the *Ward joint* described on p. 146.

In 1888 a 6-in. line of cast iron pipes with *Duane joints*, 11,760 ft. long, was laid to North Brother Island.* At the point of crossing, the Manhattan shore slopes off at an angle of more than 1 to 1, the depth of water being in one place 90 ft. The tidal currents have a velocity of 6 to 8 ft. per second, and slack water lasts at times for less than 15 to 20 minutes. These circumstances, and a large volume of commerce, made the pipe laying very difficult. It was accomplished successfully in three days at a cost of $16,245, amounting to about $9.80 per ft. The leakage in this pipe-line, as recorded by water meters, was only about 0.3 cu. ft. per minute — less than ½ per cent of the capacity of the main.

The submerged main to Blackwell's Island, described above, was carried away by anchors of vessels, and was replaced in 1891 by a line of pipes, 12 ins. in diameter by 9 ft. long, each weighing 1950 lbs. This line, which was about 1050 ft. long, cost $11,528 — about $11 per ft. When tested, the leakage in this pipe-line was found to amount only to 3½ gals. per minute.

A similar line of 12-in. pipes, 1266 ft. long, was laid in 1891 to Ward's Island at a cost of about $9.30 per ft., and another 12-in. submerged pipe, 885 ft. long, was laid to Randall's Island in 1896, at a cost of about $8.70 per ft.

In 1899 a submerged 36-in. pipe, 1248 ft. long, was laid across the Harlem River from the Fordham Road to 209th Street on Manhattan

* Proc. Municipal Engineers City of New York, 1904, p. 135.

Island.* At the point of crossing the river is 1200 ft. wide. To accomodate the demands of commerce, a trench had to be dredged for the pipe to a depth of 25 ft. below mean water. Each pipe measured 13.1 ft. over all, to lay 12 ft. The outside diameter of the hub was 5 ft., and each pipe weighed 13,725 lbs. The wrought iron band weighed 663 lbs., making a total weight of 14,388 lbs. per complete pipe length. The cost of furnishing and laying the pipe amounted to $26 per ft., exclusive of the dredging and other work involved.

The pipe was laid from a strong float, 97 ft. long and 35 ft. wide, having a steam derrick for handling the pipes, blocking for holding about five lengths of pipe and a launching way at one end, with steel cables reaved through heavy blocks, attached to the end of the float and to anchors on the shore. The cables were used to retard too sudden launching of the pipe. A couple of heavy anchors, attached to the bow, steadied the float against the tide. As each joint was completed, the steel cables were slackened, and the weight of the pipe pushed the float towards the other shore, as one length of pipe after the other was launched. The pipes were provided with Duane joints, each of which required about 1100 lbs. of lead. According to the specifications, the lead of each joint had to be poured in one operation. This was accomplished by tapping a short nipple of $1\frac{1}{2}$-in. pipe in the lead pot, and placing an ordinary cut-off valve on the other end. The hot lead was conveyed through a $1\frac{1}{2}$-in. pipe to the joint that was to be run.

151. Submerged Pipe in New York Harbor. — In connection with the great aqueduct which brings a supply of water from the Catskill Mountains to the City of New York, a line of cast iron pipes, 36 ins. in diameter and about 9800 ft. long, was laid from Long Island to Staten Island across the Narrows, a strait forming the entrance to New York Harbor.† The pipes were placed in a trench which was dredged deep enough to provide an 8-ft. covering, where the pipe-line crosses the two main channels, and which was 30 ft. deep across the anchorage ground. The maximum natural depth of water on the line of the crossing is about 60 ft., but was about 72 ft. to the bottom of the dredged trench. The bottom of the harbor is sand of various degrees of fineness, with some gravel in places, overlaid with silt and sewage sludge, brought down by the river. The dredging and pipe-laying were made very difficult on account of the swift currents in the Narrows — running ordinarily about 3 miles per hour, and changing with the tides — and the many ships of all varieties, passing through the Narrows. In the spring and winter

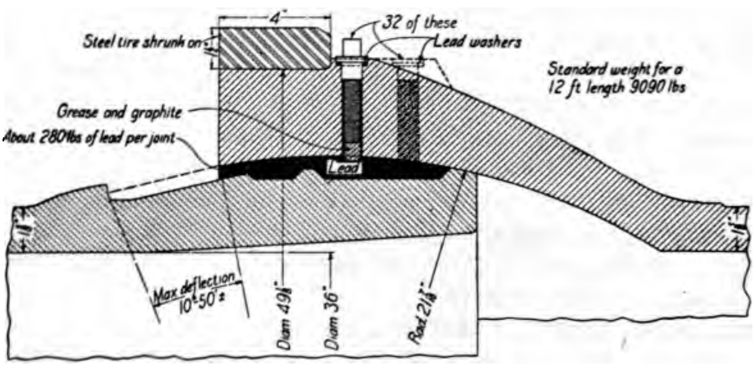
* Proc. Municipal Engineers City of New York, 1904, p. 137.

† *Eng. Record*, Sept. 20, 1913, p. 317; *Eng. News*, April 30, 1914, p. 946; *Scientific American*, Oct. 10, 1914, p. 300.

the work was often rendered especially difficult by the large cakes of ice that are brought down by the Hudson River.

Owing to the great expense that would be involved in repairing this pipe-line, great pains were taken to make the work as perfect as possible. The minimum covering of 8 ft. over the pipes was based upon experiments tried with a variety of large anchors, and upon information obtained from divers, who had recovered lost anchors in the harbor. The upper part of the refilling over the pipes was selected with especial regard to its resistance to the penetration of anchors.

The pipes, made generally in lengths of 12 ft., were joined by flexible ball-and-socket joints, which were designed after much careful study and numerous experiments. Fig. 100 shows the joint adopted for this work. The hub is strengthened by shrinking on it a steel ring, according



NARROWS SIPHON
36-INCH FLEXIBLE-JOINTED CAST-IRON PIPE
FIG. 100.

to the plan first proposed by James C. Duane (p. 146). It is turned and ground to a sphere, in which the variation from perfect sphericity does not exceed 0.006. The spigot is formed with a collar, turned to fit the bell accurately, and is provided with ridges, to give the lead a good hold. About 330 lbs. of lead are required for each joint.

Experiments were made for finding the best material for filling the lead space of the joints. Caulking, even with a pneumatic hammer, does not consolidate lead more than ⅝ to ¾ in. from the face of the joint, and this effect would be almost completely nullified by the moving of the joint. Lead wool was found to make joints which were perfectly water-tight, even under great hydrostatic pressure, but these joints were too rigid to be used in the method of pipe laying that was adopted. It was finally decided to fill the joints with good lead in the usual manner, and then to force cast lead pellets through the holes of gib-screws (tap-bolts), in order to fill the shrinkage space between the poured lead and the inner side of the hub. This was made possible by the fact, found by experiments, that lead will flow readily, even under the comparatively low pressure of 700 lbs. per sq. in., if steadily applied. This idea, which was suggested by one of the workmen, was obtained from a screw joint for high pressure steam pipes, patented by Cornelius A. Folly, of New York, in 1891. Thirty-two holes, arranged in two staggered rows,

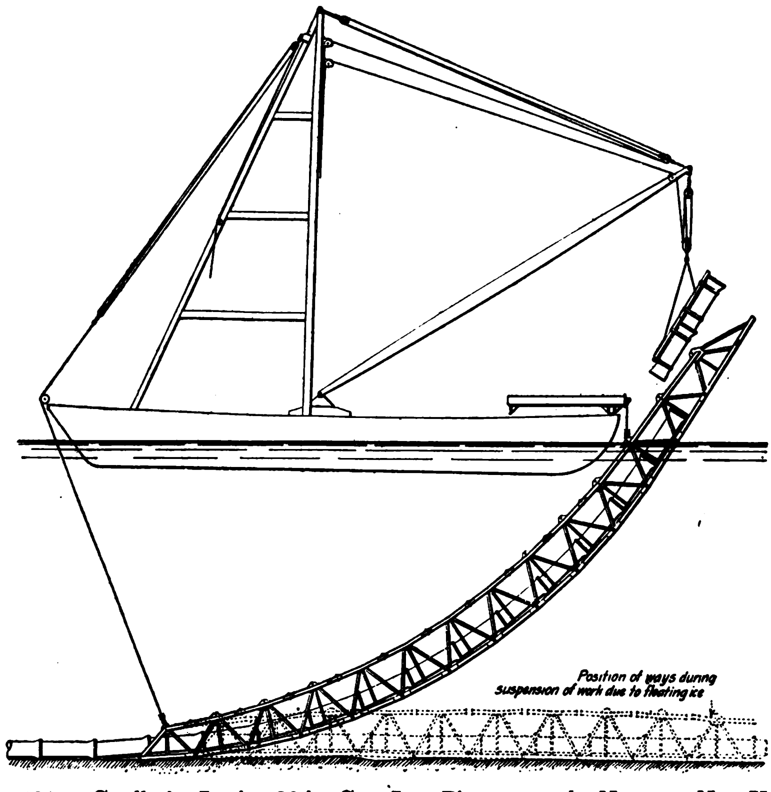


FIG. 101.—Cradle for Laying 36-in. Cast Iron Pipe across the Narrows, New York.

were provided for inserting the cast lead pellets, each of which measured $\frac{9}{16}$-in. diameter by $1\frac{3}{4}$ ins. in length. About 22 lbs. of pellets were required for each joint.

The pipes were protected by a bitumastic enamel, a superior coating, made in England and sold in America by the American Bitumastic Company. This enamel has proved to be very satisfactory, even when it is over 20 years old, for protecting iron and steel in sea water.

The joints were all made above water, and the pipes were lowered gradually on a launching cradle or skidway into the trench (Fig. 101).

The pipes were joined on the scow as follows: The inside of the hub was coated with graphite. The spigot of the next pipe was then inserted and centered and the joint poured. Lead pellets were then forced by compressed air through the holes provided for this purpose, three in each of the back holes, three in each of the front holes and then one in each of the back holes. A mixture of grease with about 10 per cent of graphite was forced into each hole, and the holes were then plugged with tap-bolts and lead washers. The joint was then deflected 5 degrees, and tested under hydrostatic pressure of 100 lbs. per sq. in., before the pipe was launched from the scow.

The joint-testing apparatus (Fig. 102) consisted of two steel bulkheads, spaced apart and kept from spreading by ten $1\frac{1}{2}$-in. bolts. Each bulkhead was fitted with special rubber gaskets and deflated rubber tubular rings, similar to automobile tires. The apparatus was provided

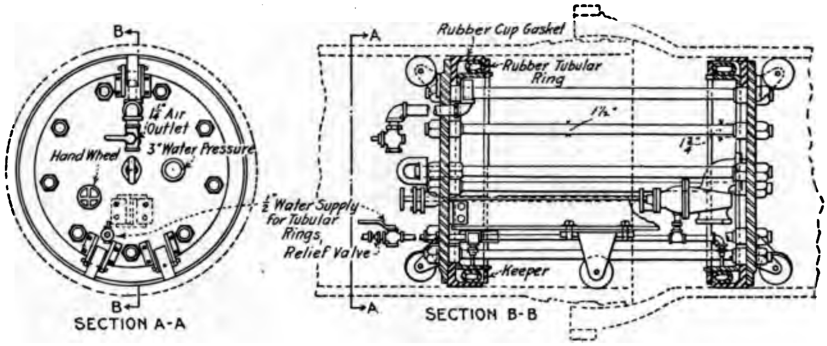


FIG. 102. — Joint Testing Apparatus.

with wheels and is rolled inside of the pipe. The distance between the bulkheads was sufficient to span the flexible joint. By means of water pipes that were connected to the tubular rings, these rings could be inflated so as to fit tightly against the inner surface of the pipe. A relief-valve permitted the deflating of the rings, when the apparatus was to be moved. The water-tightness of the joint was tested by forcing water into the space between the bulkheads, a relief-valve being provided for the escape of the air from this space.

During the laying of the pipes, the line, which is to be under a pressure of about 95 lbs. per sq. in., was tested in sections under an air pressure of 80 lbs. per sq. in., and submarine caulking was resorted to, where required. The contract obliged the contractors to lay the pipe-line so that the maximum rate of leakage would not exceed 40 gals. per joint per day.

The skidway used in laying the pipes had on each side a steel bridge truss, whose panels were connected to each other at an angle of 5 degrees, so as to produce a curved launch-way. Its lower end dragged in the trench. As the skidway was very heavy, a buoyancy cylinder was fastened to its lower end, to keep it from sinking deeply into the mud.

The scow, which measured 40 by 125 ft., was kept in position by 10 anchors, fastened to cables, 1000 to 1200 ft. long, which led to a system of power-operated drums. The scow carried a 70-ton derrick, which was used for handling the pipes.

The bids received for the unusual work of laying these pipes varied from $996,862 to $1,590,000. The contract was awarded at the former figure to the Merritt and Chapman Derrick and Wrecking Co., of New York.

152. Submerged Pipes in Boston, Mass.* — In 1850 a line of 20-inch cast iron pipes was laid across Chelsea Creek, a tidal estuary, about 1425 ft. wide at high tide, and 650 ft. wide at low water. The average

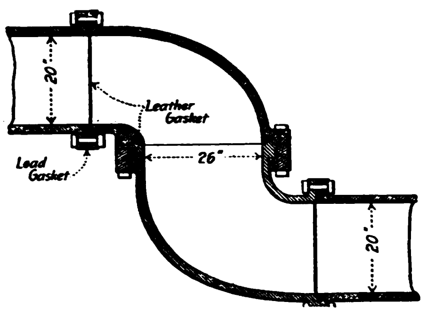


FIG. 103. — Joint of Chelsea Siphon Pipe.

range of the tides is 10 ft., and the maximum depth of water, at the point selected for the crossing, is 25 ft. The river-bottom consists of clayey material, overlaid by about 2 ft. of mud. On either side of the channel there are mud flats.

The pipes were $\frac{7}{8}$-in. thick, except across the channel, where the thickness of metal was increased to $1\frac{3}{4}$ ins. They were cast in lengths of 9 ft., and were provided with flanged joints. The pipes were supported on the mud flats by pile bents, placed 8 or 9 ft. from center to center. Each bent had two piles, and was capped with heavy timbers. Across the channel the pipe was laid in a dredged trench, the top of the pipe being 5 to 6 ft. below the bottom of the river. This part of the line formed a series of offsets, each being composed of three pieces of pipe. At each end of the section, a flexible joint of a peculiar pattern (Fig. 103) was placed. This joint consisted of two quarter bends, whose

* *Eng. News*, Feb. 28, 1901, p. 146.

diameters increased gradually from the diameter of the pipe (20 ins.) to 26 ins. where the two bends were joined to each other. At this joint, one of the bends had a flange, 6 ins. deep, which was bored for eighteen $1\frac{1}{4}$-in. bolts. The flange of the other bend had no holes. A heavy circular ring was slipped over the latter bend, and bolted to the flange of the former bend, a rubber gasket being placed in the joint. The ring was recessed to fit easily over the flange of the bend, without holes, so as to permit the bend to be turned in one plane. When the pipes provided with these joints were being laid, they were flexible at the joints in a vertical direction, but perfectly rigid horizontally.

On the flange at each end of the flexible joint, formed by the two bends, and on the two flanges of the middle pipe of each 3-pipe section, a rectangular lip, $\frac{3}{4}$ in. wide by $\frac{1}{2}$ in. deep, was cast, its inner side being a prolongation of the inside of the pipe. On the flanges of the other two pipes of each pipe section, and in a position corresponding to the lip, there was a rectangular groove, $\frac{3}{4}$ in. wide by $\frac{1}{4}$ in. deep. Thin leather gaskets, covered with white lead, were placed in these grooves, before the pipes were bolted together, as the lips were $\frac{1}{4}$ in. longer than the depth of the grooves, a smaller annular space was formed at each joint, and was filled by pouring lead.

In 1900 this pipe-line was taken up and replaced by a line of 24-in. mains.* The pipe was found to be badly tuberculated. In some places, the entire inside of the pipe was covered by tubercles, about 1 in. thick. Where the pipe had been covered with clay, after being laid in the trench, the outside of the pipe and the bolts and nuts were in excellent condition. The nuts could be easily turned with an ordinary wrench. Above low water, the pipes were badly corroded, and could be cut to a depth of about $\frac{1}{4}$ in. with a pen knife.

A large scow, 25 ft. wide by 75 ft. long, having a flush deck, was used for removing the old pipes and laying the new line. It was provided with two stiff-legged derricks and a curved cradle (Fig. 104), which was placed on one side of the scow and formed a launch-way, down which the pipes were lowered. This cradle was well braced and trussed. It was hung by wire ropes from the larger of the two derricks, the other derrick being used to raise or lower the tail end of the cradle, so that it might at all times be tangent to the bottom of the dredged trench. The tackle was so arranged that the whole cradle could be raised or lowered vertically, or tipped at any angle in a vertical plane. A wooden shoe, about 4 ft. long, was fastened to the tail end of the cradle, for the purpose of making the cradle rest more evenly on the bottom. It also served for smoothing off small irregularities along the bottom of the trench. The

* Jour. Assoc. Engr. Soc., 1901, XXVI, p. 191.

pipes were laid by means of the scow and its cradle in the following manner:

The cradle was filled with pipes, the joints of which were leaded and caulked. The scow was then brought at high tide into the proper position over the pipe trench, which had previously been dredged out. The tail end of the cradle was then tilted down, until it rested on the bottom of the trench, but at such a depth that the end of the pipe would be exposed at low water. This end of the pipe was then securely anchored, and the scow was pulled ahead about 12 ft., causing the pipe to be pulled down the cradle.

By means of a small boom-derrick, another pipe was then placed on the cradle, and the process was repeated until all of the pipes had been placed on the cradle and slid down into the trench.

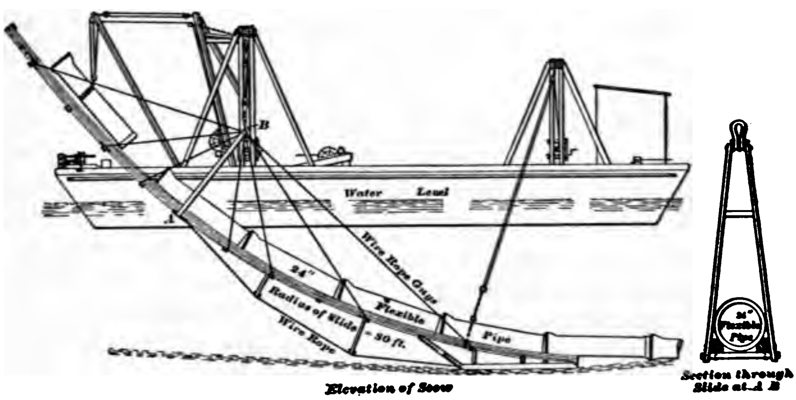


FIG. 104. — Chelsea Cradle.

During the work, the pipes were stored on a large lighter, which was moored a short distance away. For the immediate work, 10 or 12 pipes were stored on the pipe-laying scow and on a smaller scow which served as a tender. The laying of 54 pieces of submerged pipe occupied two weeks. Above low water, the pipes were laid on the same pile foundation that had supported the old pipe-line. Although these piles had been in use for a long time, they were found to be perfectly sound below the mud lines, and only slightly decayed on the outside of the piles, above the mud. In laying this part of the pipe-line, the pipes were usually made up on the scow in four-pipe-sections. The scow was then floated into position at high water, and the pipe lowered onto the caps of the piling. At low water, the pipe section was pulled by means of tackle and falls so as to join the previously laid section properly, and the joint between them was leaded. After all of the pipes had been laid, the joints, both above and below low water, were thoroughly caulked, and the line was then subjected to a hydrostatic test of 84 lbs. per sq. in. for an hour. The leakage recorded by the flow through a $\frac{3}{4}$-in. water meter was barely perceptible.

For the portion of the line that was above low water, ordinary bell-and-spigot pipes, 0.95 in. thick, were used. Below low water, the pipes were made 1.25 ins. thick, and provided with ball-and-socket joints, similar to those used in the Mystic River crossing, described below.

The cost of the work estimated from the force account kept by the inspectors was about as follows:

For removing the old pipe and laying the new pipe with spherical joints the cost was $8.25 per lineal ft. For removing the pipes above low water and laying the new pipes, the cost was $2.25 per lineal ft. These figures do not include the cost of the pipe, nor the salvage of the old pipe. A rental value for the use of the plant was included and the cost of the dredging was estimated from the rental value of the dredges.

153. Mystic River Crossing.* — In 1897 two lines of 36-inch cast iron pipes were laid, 5 ft. 9 ins. between centers, across the Mystic River, just east of the bridge on Middlesex Avenue, and connected by means of Y-branches with 48-in. pipes, which had previously been laid on each shore. At the point of crossing, the river is a tidal stream, about 1100 ft. wide at high water, and about 300 ft. wide at low water. The average range of the tides is about 10 ft., and there is a depth of about 9 ft. of water in the channel at low tide. The river-bed at this point consists of a stratum of sandy silt, overlaid by 10 to 20 ft. of mud, except at one of the shores, where sand and gravel are found.

The pipes, which are 1.65 ins. thick, were laid in a dredged trench, which was about 35 ft. wide at the top, and had an average depth of about 8 ft. below the surface of the mud, the lowest point dredged being about 16 ft. below mean low water. Piles were driven in this trench to give the pipes a firm support. Two piles were placed in each bent, 6.3 ft. between centers, crosswise of the trench, the bents being 12.1 ft. apart, and in such a position that each pipe, when laid, would have bearing on a pile bent, about 4 ft. back of the face of the bell of the pipe, except at the spherical joints, where an extra bent was driven, in order that there might be a support on each side of this joint. The piles were cut off, and 10 by 10-in. caps were bolted on under water by a diver. After the pipes had been laid, the trench was refilled to a height of 2 ft. above the tops of the pipes.

* Jour. Assoc. Eng. Soc., 1901, XXVI, p. 193.

The following five different kinds of pipe were used:

	Length, feet	Weight, pounds	Cost per ton	Lead per joint, pounds	Depth of lead in joint, inches
Spherical bell with spherical spigot	12.53	8260	$23.90	248.0	8
Spherical bell with bead spigot	12.17	8140	23.90	248.0	8
Grooved bell with spherical spigot	12.59	8040	23.90	81.5	3
Grooved bell with bead spigot	12.10	8210	17.90	81.5	3
Grooved bell with taper spigot	10.10	8030	22.90	81.5	3
Sleeves	....	....	40.00	128.3	4

The pipes were connected by spherical ball-and-socket joints of the *Ward* type (Fig. 81, p. 145), designed for a maximum deflection of 1 in

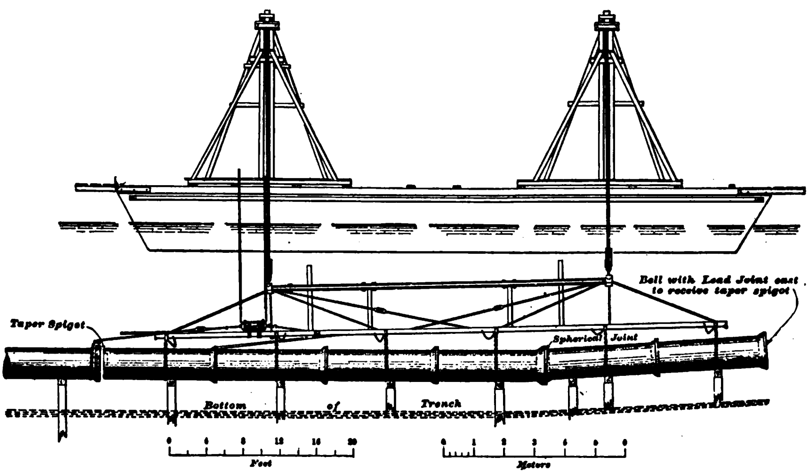


FIG. 105. — Scow for Laying Mystic Siphon Pipe.

10 in any direction, without the spigots leaving the bell. In these joints, the lead always remains in the spherical bell, while the raised portion, cast in the spigot end of the next pipe, and turned truly spherical, plays against the stationary lead in the pipe-bell, when the joint is deflected. At the base of this raised portion is a stop, against which the face of the bell is pressed, when the maximum deflection of the joint is reached, while the spherical bell is a raised ring, turned to a true circle, which presses tightly against the raised portion cast on the spigot, and prevents lead from running into the pipe when the joint is run.

The pipes were laid from a flat scow, 23 ft. wide by 70 ft. long, on which two stiff-legged derricks were erected. A straight truss, about

75 ft. long, was suspended from the derricks, on one side of the scow (Fig. 105), in such a manner that a section of pipe, fastened to the lower chord of the truss would hang parallel with, and just clear of, the side of the scow. A smaller scow, loaded with gravel, was fastened to the other side of the large scow, and served as a counterweight to balance the pipe on the truss.

The pipes were connected, usually in sections of 6 pipes each, on a temporary wharf. At one end of a section, a pipe with grooved bell and taper spigot was placed, the other pipes being usually of the ordinary pattern. Into the bell end of the last pipe of each section, the taper spigot of another pipe was temporarily inserted, and the joint was run with lead. After the joint had cooled, this spigot was pulled out, leaving the lead joint in the pipe. The pipe with this taper spigot would be the first pipe used in the next section, and when this section was put in place, this spigot would again be fitted into the lead joint from which it had been pulled.

After a section of pipe was fastened to the truss by chains, the scow was brought into the proper position by means of winches and anchors. The truss, with the pipes attached to it, was then lowered to the piling, and placed, as directed by a diver. A hydraulic cylinder was attached to the end of the truss, near the pipes already laid. Its piston rod was provided with a hook at the end. A chain, fastened back of the bell of the last pipe laid, was attached to the hook, and by forcing oil into the hydraulic cylinder, the truss was drawn forward, and the spigot of the first pipe attached to it was forced into the bell of the last pipe laid. An iron collar was attached to the bell, and served the double purpose of enabling the diver to insert the spigot with ease, and of keeping the lead joint in the bell from being forced out of place, by the spigot being carelessly entered. Wooden bulkheads were kept in the ends of the pipe sections, until just before the spigot was forced home, in order to keep mud and foreign bodies out of the pipes.

The pipes were laid across the river in a straight line, spherical joints being used where vertical deflections occurred. Pipes with these joints were built into the sections, like the other pipes, the joints being deflected to fit the position in which the pipes were to be placed. After all the pipes had been laid, the joints were thoroughly caulked by a diver, and the pipes were then tested by air under a pressure of 25 lbs. per sq. in., and then by water pressure for leakage, before the trench was refilled. Manhole pipes, for allowing access to the interior of the pipe-lines, were placed on each line, on both sides of the river. They were closed by suitable covers.

The work described above was performed in 1897. The total cost, including pipe, labor, materials, and an allowance for the use of the tools and plant, was about $13.25 per lineal foot, of which $6.75 per lineal foot was paid for the pipes.

154. Submerged Pipes in Portland, Ore. — In connection with the water supply of Portland, Ore., three lines of submerged pipes were laid across the Willamette River.*

The first line, which consisted of 28-in. cast iron pipes, in lengths of 17 ft., was laid in 1894. All of the joints were flexible and were made according to Fig. 85 (p. 148). The pipes were laid in a trench, 10 to 12 ft. wide, and 8 to 23 ft. deep, which was dredged out by means of a stern-wheel steamboat, fitted up with buckets attached to an endless chain, and operated from one side of the boat. The dredge was kept in line by means of a row of piles, driven about 35 ft. apart (except in the channel), at a convenient distance up-stream from the pipe-line. These piles were used afterwards for keeping the pipe-laying barges in line.

The specifications required the ball-and-socket joints to have exact spherical surfaces of given diameters. After the pipes had all been accepted by the inspector at the foundry and delivered, it was found that many of the balls and sockets were not truly spherical, the variations from the required radius at the center, measured near each edge, being from −0.012 to +0.012 in. When a few lengths of pipe were joined and tested under the low pressure of 10 lbs. per sq. in., considerable leakage occurred, but when the pressure was raised to about 45 lbs., the leakage ceased entirely. In order to avoid delay, the pipes were, therefore, accepted and laid.

In the shallow water near each bank of the river, the pipes were laid by suspending them from pile-bents by means of long $1\frac{1}{4}$-in. screw-rods, and lowering them gradually into position, by turning the nuts. A pile-bent was provided for each length of pipe. In deep water, the pipes were laid by means of a launch-way or cradle (Fig. 106), constructed in the form of an inverted bow-string. One end was supported and pivoted between two barges, placed 10 ft. apart, while the other end was suspended between the barges at the rear by a long rod, which was always so adjusted as to permit the cradle merely to touch the bottom of the trench. The cradle † was designed to have a factor of safety of 10 under ordinary loads, so as to make some allowance for shocks to which it was subjected from heavy swells from wind and passing steamboats. In order to provide a continuous slide to the bottom of the trench, an apron shod with heavy plate iron was hinged at the first panel of the

* Trans. Am. Soc. C. E., 1895, XXXIII, p. 257.

† *Ibid.*, p. 265.

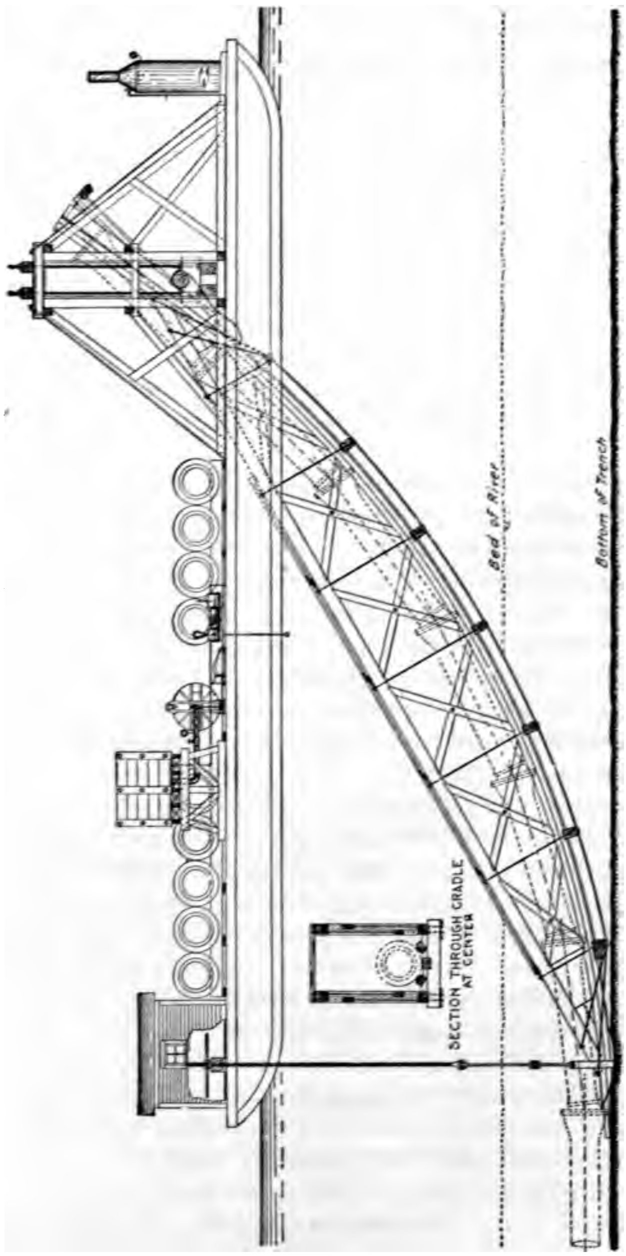


FIG. 106. — Cradle for Laying Submerged Pipe at Portland, Ore.

cradle. It answered its purpose very well, even on the east side, where many boulders were encountered.

The pipe-laying force consisted of 1 foreman, 2 engineers, 6 men for testing the pipes and cleaning the ball ends, 1 man melting lead, 6 men pouring lead, caulking and bolting — a total force of 16 men. An expert diver was employed continuously. In spite of much rainy weather, 100 pipes were laid in 20 days, an average of about 80 lineal ft. per day. Towards the close of the work, as many as 6 to 8 pipes were laid in 10 hours.

The line was tested by hydraulic pressure for about every 200 ft. laid. During these tests, the diver inspected each joint carefully, and the pipes in the trench. Leaks were easily discovered, and were promptly stopped by caulking. Very little caulking under water was required, as the pipes were practically tight as laid. After the whole line had been completed and connected on each bank with the 32-in. main supplying that part of the city, the pipe was subjected to thirteen tests, including the final test, prescribed by the specifications. In the last test the pipe was subjected to a continuous pressure of 200 lbs. per sq. in. for 24 hours. The leakage found during this test, which included, also, that of 600 ft. of the 32-in. main, was only $\frac{3}{4}$ gal. per minute. During two of the tests, a pipe was broken each time. These breaks were, doubtless, caused by the wedging of the ball in its socket, the end pressure concentrating an enormous strain on a comparatively small portion of the socket.

155. *A second line of pipes* was laid across the Willamette River in 1898. It is 2042 ft. long, and consists of 24-in. lap-welded steel pipe, $\frac{3}{8}$ in. thick. The line has 58 flexible joints of special design (Fig. 86, p. 149), which are placed at intervals of 20–40 ft., depending upon the alignment and grade of the pipe trench. The methods and appliances adopted for this work were similar to those used in laying the 28-in. pipe. A trench was excavated for the pipe by a ladder dredge, and a cradle or launch-way was used in laying the pipe.

The location selected for this pipe-line was parallel with, and about 10 ft. from, that of the 28-in. main, until the harbor was reached. The line was then deflected south, on each side of the river, until a line parallel with the 28-in. pipe-line, and 100 ft. up-stream from it, was reached, on which the remaining part of the pipe-line was laid. The grade adopted was practically that of the first pipe-line.

After the work had been completed, a test pressure of 170 to 180 lbs. per sq. in. was applied to the pipe-line. The joints were not found to be absolutely tight, but the total leakage shown in a final test amounted only to 10,080 gallons per day, equal approximately to 7 gals. per minute, or 8 gals. per lineal foot of joint per day, under the pressures given, which were about 50 per cent in excess of the normal pressure.

The cost of performing this work, including two joints and 72 lin. ft. of pipe not used, was as follows:

COST OF SUBMERGED PIPE

60 flexible joints, $260 each	$15,600
1938 lin. ft. of 24-in. pipe, $8.50 per lin. ft.	16,472
15,332 cu. yd. dredging, $0.40 per cu. yd.	6,133
2041.6 lin. ft. pipe-laying, including joints, $1.77 per lin. ft.	3,613
Total	$41,818

During 1898 the Water Department paid $1.75 to $2.00 per day for labor.

156. *Submerged 30-inch Steel Pipe.** — A few years after the 28-inch and 24-in. pipes had been laid across the river, these pipes had to be lowered about 14 ft., so as to provide a depth of 30 ft. at low water.

It was decided to take up the pipes and to relay them in new trenches. Before this was done, a new line of 30-in. steel pipes was laid across the river, in order to maintain the water supply. After the new line of pipes had been laid, it was decided only to relay the 24-in. steel pipes, and to store the 28-in. pipes for future use.

The *third pipe-line*, which is 30 ins. in diameter and about 4300 ft. long, consists of lap-welded, galvanized and asphalted steel pipes where it is submerged, and of standard cast iron pipes on both shores. The steel pipe was made of $\frac{7}{16}$-in. plates, and was provided with flanged joints, which were bolted together with $1\frac{3}{8}$-in. bolts with hexagonal nuts. The ball-and-socket joints were rotated and tested before leaving the shop. Each joint weighed about 5700 lbs. Twenty-seven flexible joints were inserted in the steel main, at intervals of about 60 ft.

The pipes were laid by means of three derrick barges and two common deck barges, without the use of any special crib, cradle or tools. The pipe laying was begun at the east shore. After the inclined section and two additional lengths of pipe had been laid, the work was carried on as follows: One derrick held the westerly end of the pipe above the water, while three or four lengths of pipe with one flexible joint, previously bolted together, were joined to it. The derrick then lowered the pipe until only the westerly end was above the water. The total weight handled amounted to 8 to 12 tons, before being submerged. The slings were marked, and care was taken during the lowering that the flexible joints were not made to deflect more than 11 degrees. The rear slings were then slipped, and the operations were repeated. As the pipe was

* *Eng. News*, Feb. 10, 1898, p. 100.

nearly buoyant, it had to be filled with water during the lowering, in order to keep the strain on the derricks as uniform as possible.

The flow through the pipe-line is controlled on each shore in a concrete gate-chamber, having a top of reinforced concrete with standard manhole covers. The valves used are extra heavy 30-in. double disc gate-valves with by-pass and indicator. Two 4-in. relief-valves are placed at each end of the pipe-line at the gate chambers, and set to blow off at a pressure of 135 lbs. per sq. in., which is 15 lbs. above the average working pressure at the points mentioned.

The 30-in. cast iron shore connection cost $11.10 per ft. and the cost of the 30-in. steel pipe was $14 per ft. Each flexible joint cost $345. The total cost of the work was as follows:

SUMMARY OF COSTS

Dredging	$20,318
Steel pipe	29,135
Cast iron pipe and pipe laying	65,747
Labor and engineering	4,808
	$120,008

On the completion of the work, the whole new pipe-line was tested under a pressure of 175 lbs. per sq. in. Under the normal pressure of 150 lbs. per sq. in., the leakage was found to amount to about 30,000 gals. per day. An investigation showed that this loss did not come from any large leak but from a number of small ones. In a test made about a year and a half after the completion of the work, the leakage was found to have been reduced to about 5000 gals. per day. The loss of water continued to diminish and amounted in 1914 only to 1000 gals. for 24 hours.

157. Pipe Across the Passaic River, New Jersey.* — In 1896 the 48-in. pipe-line of the East New Jersey Water Company, which supplies Newark, N. J., with water, had to be carried across the Passaic River at Belleville, N. J. At the point where the crossing was to be made, the Passaic River is a tidal stream, having at high water a width of about 600 ft. and a depth of about 15 ft. The tide has a range of 4 ft., and there is daily considerable navigation on the river.

The pipe-line, which was to be extended across the river, has a daily delivery of about 40,000,000 gals. It is composed of riveted steel pipes, which are subjected to a pressure of about 350 ft. head at the river. The ordinary kind of flexible joint, filled with lead, could not be used for the submerged part of the pipe-line, for the reason that such joints, even when well made, and water-tight at the outset, would soon wear out or

* Jour. Assoc. Eng. Soc., 1901, XXVI, p. 216.

cut out, under the great pressure the pipe had to sustain, and cause a great loss of water.

The crossing was made by laying, in a trench dredged for this purpose, seven parallel lines of 18-in. lap-welded steel pipes, connected by screw joints, and loaded by cast iron reinforcements at each joint. While such a pipe-line with screw-joints is certainly stiff, a line 600 ft. or more long, made up of 20-ft. sections, has considerable flexibility. Each line of these 18-in. pipes, after having been joined on shore, in sections 200 ft. long, with a temporary cap at each end, was dragged across the river by an ordinary hoisting engine, placed on the opposite bank. This operation was made very easy, as the pipes had been so loaded, as to have only a slight excess of weight over the power of flotation. After the first section of 200 feet had been hauled out into the river, the in-shore cap was taken off, and a second section was connected to the first one. The two sections were then pulled forward, and the operation of attaching additional sections was repeated, until the end of the first launched section appeared above water on the opposite shore.

After the crossing had been made, the flow through the submerged pipes was tested by means of two Venturi meters, one placed on each bank. The pipes were found to be perfectly water-tight. There is no doubt that this method of placing pipes in a river-bed could be used for mains of considerably larger diameters.

158. Submerged Pipe at Escanaba, Mich.* — A wrought iron suction pipe, 12 ins. in diameter and 2000 ft. long, was laid across the Escanaba River in the winter of 1898–99, by putting the pipe together on the thick ice that covered the river, and then cutting the ice from under the pipe, so as to permit the pipe to settle gradually to the river-bed. At the site of the crossing, the river has a depth of 10 ft. The pipe was laid in the usual manner on blocks which were placed on the ice. After the joints had been caulked, cross-timbers, supported by blocks, were placed over the pipe at each joint, a rope was fastened to the cross-piece, and its free end was passed under the pipe and secured temporarily to the cross-timber. The ice was then cut away gradually from under the pipe, and the pipe was carefully lowered by small distances, so that when the last joints were caulked, the part near the bank was already on the bottom. Three derricks were used in lowering the pipe to the river-bed, the rear one being carried around to the front, as fast as each successive joint was made.

After the pipe had been laid on the river bottom, it was sunk by means of a water jet about 2 ft. into the river-bed, which consists of sand. The power for the water jet was furnished by a 5 horsepower pump, which

* *Eng. Record*, June 24, 1899, p. 72.

forced about 1000 gals. per minute through a 2-in. pipe, provided with a suitable nozzle.

The work of joining and lowering the pipe was done by 20 men in 5 days at a cost of about $200.

159. Submerged 6-foot Concrete-jacketed Steel Pipe.* — A riveted steel pipe, 6 ft. in diameter, forming part of the water supply of Jersey City, N. J., was laid in 1902 across the Hackensack and Passaic rivers. At the point where the Hackensack was crossed, the river is about 575 ft. wide and 45 ft. deep. The pipe was put together on shore in 32-ft. sections, the ends being closed by conical-shaped bulkheads, and the sections were pulled across the river by means of a steam engine, located on the opposite shore. While the pipe was being hauled across the river, it was braced on the inside by timbers, in order to prevent its being collapsed by the water pressure. This bracing formed the load needed to keep the pipe on the bottom, until it was laid from shore to shore, when it was encased with concrete and loaded with rip-rap. The bracing was then removed.

The pipe was made rigid by the concrete casing which was grouted. This was done by making concrete cylinders, 6 ins. thick and 5 ft. long, in advance of the steel pipe. The concrete cylinders were slipped on over the exposed shore ends of the submerged mains, jointed with cement, and run full of grout, through a hole left in each section for this purpose. The concrete was designed to protect the pipe, to give it the necessary weight, to keep it from floating, and, also, to prevent the collapse of the pipe, when it is empty, by the outer water pressure. By the method described, the pipe was kept submerged, while being laid, and offered no obstruction to navigation.

* *Eng. News*, March 12, 1903, p. 232.

CHAPTER XI

GATES AND VALVES

160. Gate-valves are required in all pipe-lines for controlling the flow of the water. The smaller sizes are made entirely of bronze, but, in the larger gates, the body is usually cast iron, bronze mounted. In its simplest form, a gate-valve is a wedge-shaped disc, controlling the flow through a pipe. It is operated by means of a rod, provided with screw threads, which may be either an *outside stem* or an *inside stem*. In the former case, the bottom of the stem is attached to the gate, and its upper, screw-end passes through a nut which is kept by a yoke in the same horizontal plane. As this nut is revolved by means of a handwheel or lever, the screw and the gate to which it is attached rise or descend, according to the direction in which the nut is turned (Fig. 107). This arrangement has the advantage that the position of the end of the stem indicates exactly the position of the gate, and it also facilitates the lubrication of the stem. On the other hand, the stem is exposed to injury. Gates with outside stems and yokes are, therefore, only used in pumping stations and other protected places.

FIG. 107. — Gate-valve with Outside Stem and Yoke.

Gate-valves which are to be placed on street mains are always made with inside stems, as shown in Fig. 108. In this case the stem is prevented from rising or descending by means of a collar which is kept in the same horizontal plane. A nut is firmly attached to the top of the stem, and by turning it, by means of a special wrench or key, the valve stem is revolved. The lower part of the stem is provided with a screw thread which engages with corresponding screw threads, cut on the inside of the valve. As the stem is revolved in the direction required for opening the valve, it

FIG. 108. — Gate-valve with Inside Stem.

screws into the gate and raises it. By turning the stem in the opposite direction, the gate is lowered. An arrow should be cast on the top of the gate or on the nut, to show how to turn the nut to open the gate.

Instead of making the gate a solid wedge, it may be formed of two adjustable wedges, placed back to back. By suitable contrivances, the two wedges can be made to leave their seats quickly, when the gate is to be opened, and to close rapidly when the gate is to be shut. Gate-valves for pipes 18 ins. or less in diameter are usually placed vertically, and are protected by suitable gate-boxes. Larger gates are generally set horizontally, so as not to project into the street. Such gates are usually protected by masonry vaults to which access is obtained by means of cast iron manholes. Small gates may be set on flat stones, but large ones require a masonry foundation.

To illustrate the details of the construction of gate-valves, we give below descriptions of some of the principal gate-valves manufactured in America.

Fig. 109.—Eddy Gate-valve (taper-seat, double-gate).

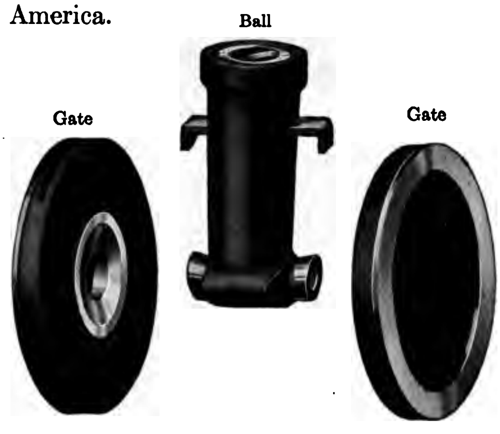


Fig. 110.—Inside Parts of Eddy Gate-valve.

161. The Eddy taper-seat, double-gate valve * (Fig. 109) consists of two gates and of a piece, called the ball. The gates (Fig. 110) are hung on two trunnions on the ball, which fit into cylindrical recesses in the gates. This arrangement allows the gates to revolve freely on the trunnions, so as not to seat always in the same position. At the base of each trunnion, there is a convex surface which fits a concave surface on the back of the gates. This permits the gates to adjust themselves to their tapered seats. Two hooks, attached to the ball,

* Manufactured by the Eddy Valve Co., Waterford, N. Y.

engage loosely in recesses on top of the gates, which are thus prevented from spreading. As the gates are center-bearing, they are forced with equal pressure at all points, as the gate-valve is being closed against its tapered seat.

FIG. 111.—Eddy Gate-valve (parallel-seat, double-gate).

Valves from 2 to $3\frac{1}{2}$ ins. in diameter are made with solid bronze gates, and do not have the hooks on the balls. Larger valves, up to 48 ins. in diameter, are made either all iron, or with iron body, bronze mounted, bronze being used in the latter case for the stem, case and gate rings, and a bronze mounted ball. The valves are made with hub, flanged, screwed or spigot ends.

162. The Eddy parallel-seat, double-gate valve (Fig. 111) consists of two gates, a carrier-wedge, a stop-wedge, and two center wedges. The gates are suspended independently from the carrier-wedge by means of two lugs cast on the back of each gate. The stop-wedge is suspended from the carrier-wedge by means of flanges. When the gate has been almost closed, the stop-wedge strikes a boss at the bottom of the valve-case. As the revolving of the valve-stem is continued, the two center wedges are forced outward between the carrier-wedge and the stop-wedge and thus force the gates firmly against their seats. As each gate is independent of the other, any obstruction on the seat of one of them will not interfere with the closing of the other.

163. The Ludlow double-gate valve * (Figs. 112 and 113) consists of two parallel gates or discs, between which two bevel-faced wedges are held by ribs on the back of the discs. The gate-stem, which is kept in position by means of a collar, screws into a brass nut in the upper wedge. When the gate is being closed, the lower wedge is stopped by a boss, when the gates are exactly opposite the openings they are to close. As the revolving of the stem is continued, the upper wedge presses against the lower one, thus forcing the gates tightly against their seats. In opening the valve, the first turn of the stem moves the upper wedge from contact with the lower one, thus releasing both gates from their seats, before they commence to rise. All grinding on the face bearings of the gates is thus prevented. The brass nut in the upper hich the stem screws, has center bearings, which prevent

factured by the Ludlow Mfg. Co., Troy, N. Y.

the gates from being canted to either side in closing and wedging. The valve works equally well with pressure on either side of the gates. They

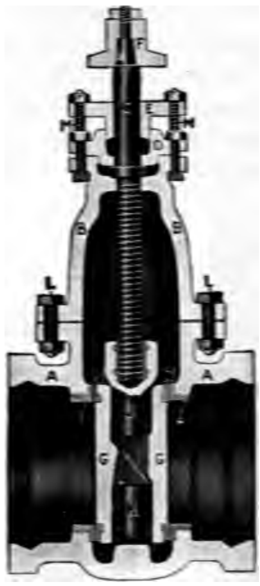


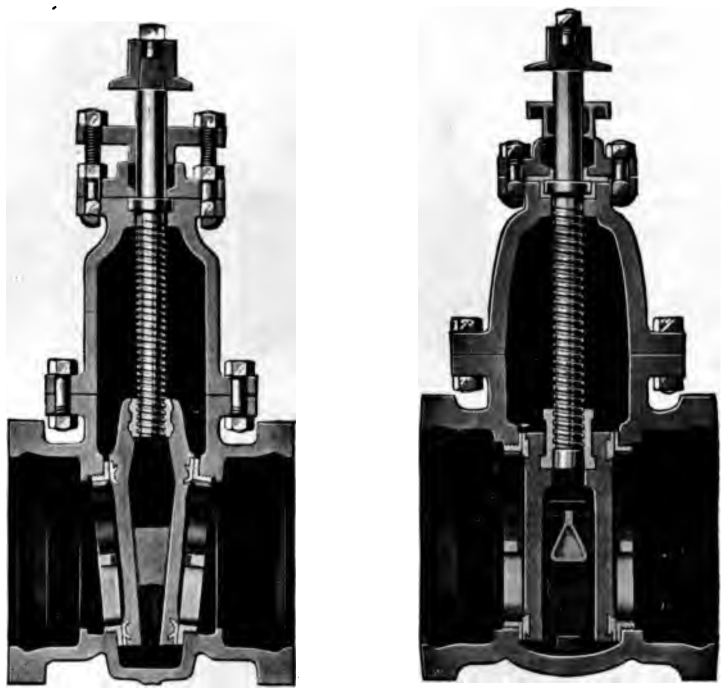
FIG. 112.—Ludlow Double-gate Valve.

A—Case.
B—Cover or Bonnet.
C—Stem or Spindle.
D—Packing Plate or Stuffing Box.
E—Stuffing Box Gland or Follower.
F—Wrench Nut.
GG—Gates.
H—Gate Rings.
I—Case Rings.
J—Top Wedge.
K—Bottom Wedge.
L—Throat Flange Bolts.
M—Stuffing Box or Follower Bolts.

FIG. 113.—Gates and Wedges for Ludlow Double-gate Valve.

are usually made of cast iron, with bronze mountings, with hub, flanged or spigot ends. The gates and seats have ground faces, to insure water-tight joints.

164. The Chapman Valve Manufacturing Company makes gate-valves for water pipe, 2 to 108 ins. in diameter (Fig. 114). These valves have iron bodies and caps, and either wedge-shaped gates or double-gate parallel seats. All water valves are bronze mounted throughout, having bronze spindle, bronze-faced wedges or gates, and bronze or babbitt seats. The valve-stems are equipped with a square iron nut on top and are easily operated. The valves are made to withstand working pressures from 50 to 300 lbs. per sq. in.



a—Taper Seat. *b*—Parallel Seat.

Fig. 114.—Chapman Double-gate Valves.*

165. The Coffin double-disc gate-valve,† for pipes 6 to 60 inches in diameter, is shown in Fig. 115. The body of the gate-valve is made of cast iron, but the valve and seat-rings, valve-stem, gland-bolts and nuts, packing-gland, stuffing-box, etc., are all made of bronze. The two discs, and a nut between them, into which the stem screws, are cast in one piece. The stem is held by means of a collar, so that it cannot rise, when being revolved. It engages, therefore, with the nut between the discs, and thus raises or lowers the valve, as the case may be.

* Manufactured by the Chapman Valve Mfg. Co., Indian Orchard, Mass.

† Manufactured by the Coffin Valve Co., Boston, Mass.

Gate-valves, 6 to 24 ins. in diameter, are usually set with the stem in a vertical position. Larger gate-valves are usually placed with their stems horizontal, the stems being moved by means of bevel gear. A mechanical graduated indicator shows the position of the valve. For heavy pressure, each gate-valve is provided with a by-pass. For a 48-in. gate-valve, the by-pass is 6 ins. in diameter. For light pressure, the Coffin Valve Co. manufactures single disc valves.

Fig. 115. — Coffin Gate-valve.

166. Rensselaer double-gate valves* provide a straightway passage having the full diameter of the pipe. They have parallel faces, moving on each other, like the blades of a pair of scissors, scraping off any foreign substance adhering to them. In closing the valve, each gate moves without friction to its position opposite its port, and both gates are then forced, by wedges between the gates, squarely against the ports. The wedges, which are entirely independent of the operating nut, and screw, act equally on both gates. The spindle being always free, is never thrown out of alignment and made to bind. In opening, only one gate is moved, until the entire wedging and locking arrangement is released. Fig. 116 illustrates the inside working parts of the gate-valves for sizes up to, and including, 8 ins. All wearing parts are bronze mounted. The nut through which the spindle revolves is solid bronze.

Fig. 117 shows the working parts for horizontal gate-valves, 10 ins. or more in diameter. These gates are provided with anti-friction *bronze rolls*, and with *bronze scrapers and side guides or wings*. The bronze tracks never corrode, or become uneven, and the bronze scrapers keep the tracks free from sediment. The bronze side guides or wings prevent gates with single spindles from *rolling* sideways, when striking any obstruction.

167. Automatic Gate-valves. — In the 40-inch pipe-line, which was laid, about 1850, from the service-reservoir at Godlay to Manchester, England, ordinary gate-valves were placed $1\frac{1}{4}$ miles apart. At three places in this pipe-line — which is $8\frac{1}{2}$ miles long — automatic gate-

* Manufactured by the Rensselaer Valve Co., Troy, N. Y.

A—Case or Body.
B—Cover or Bonnet.
C—Gate, Long Incline. Valves 10 in. and larger.
D—Gate, Short Incline. Valves 10 in. and larger.
E—Gates. Valves 8 in. and smaller
F—Stuffing Box.
G—Follower or Gland.
H Blind Nut.
I—Stem Nut.
J—Stem.
K—Case Rings.
L—Gate Rings.
M—Bolts for Stuffing Box and Followers.
N—Nuts for Stuffing Box and Follower Bolts.
O—Nut for Operating Wrench.
P—Bolts for Case and Cover Flanges.
Q—Wedges for Valves 10 in. and larger.
R—Hooks for Valves 10 in. and larger.
S—Wedges for Valves 8-in. and smaller.
T—Straps or Links.
U—Set Screws for Straps.
V—Tracks.
W—Scrapers.
X—Trunnions or Rollers.
Y—Side Guides.

FIG. 116.—Section of Valve.

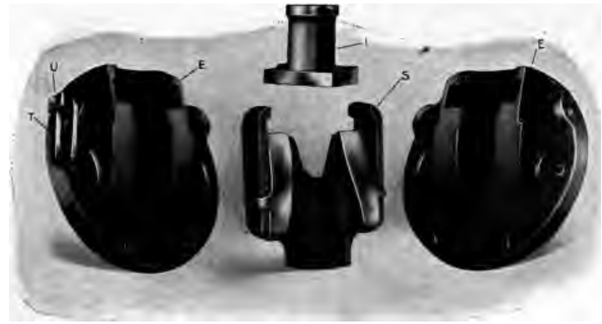


FIG. 117a.—Gates and Wedges, Valves 8 Inches and Smaller.

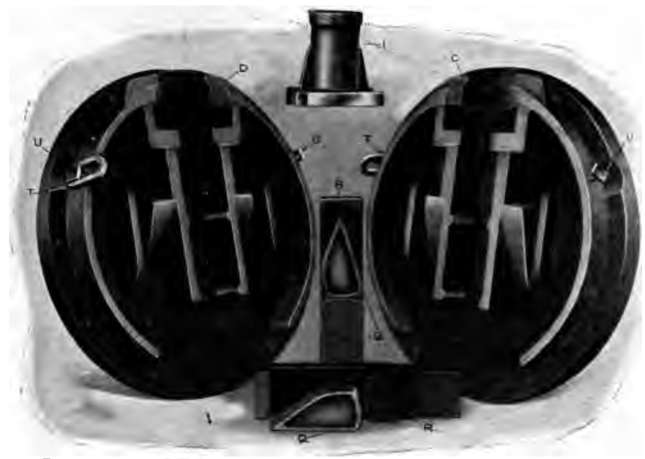


FIG. 117. Gates and Wedges, Valves 10 Inches and Larger. Rensselaer Double-gate Valve.

valves, manufactured by Armstrong and Company, were installed for the first time. Similar gates were placed in the siphons of the Thirlmere and Vyrnwy aqueducts, and on other important pipe-lines. In case a pipe breaks on the delivery side of the valves, the sudden rush of water closes the gate slowly, and causes a bell to ring in the keeper's house.

If possible, the automatic gate-valves are installed where the pressure is comparatively light, good locations being placed a little below the

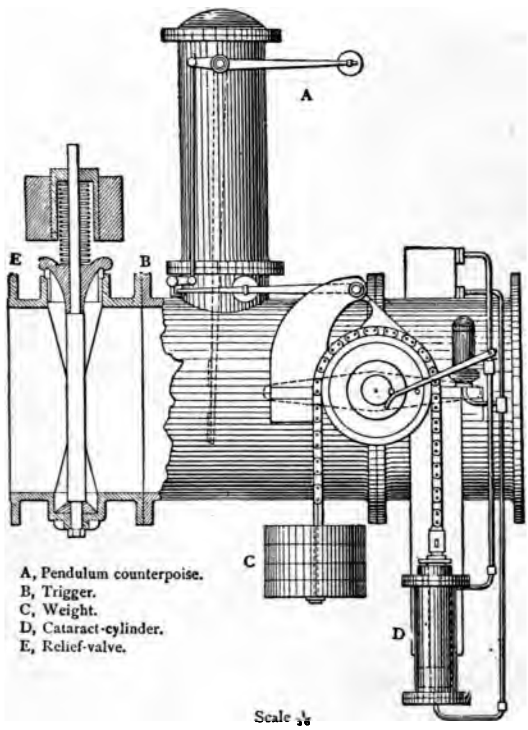


FIG.* 118. — Automatic Gate-valve.

hydraulic gradient, on the verges of deep depressions in the pipe-line, where fractures are most likely to occur. The construction of the gate is shown in Fig. 118.* A pendulum, dotted in the figure, carries a small disc within the pipe, normal to its axis, and is balanced by a weight and adjusted to hang vertically when the velocity in the main is normal. If the velocity in the pipe becomes much increased, on account of a break in the pipe-line on the delivery side of the valve, the additional pressure

* Reproduced from "Water Works Engineering," by J. H. Tudsbery Turner and A. W. Brightmore, London, 1893, p. 373.

of the water against the disc deflects the pendulum, and causes, by trigger-action, a heavy weight to be released, which closes the main throttle-valve (shown in outline in the figure) by a chain passing around the periphery of the valve-wheel. A dash-pot prevents the gate from closing too suddenly, 15 minutes time being consumed in closing a large gate-valve, the latter part of it being effected at a gradually diminishing rate. This is accomplished by throttling automatically the supply pipe of the dash-pot.

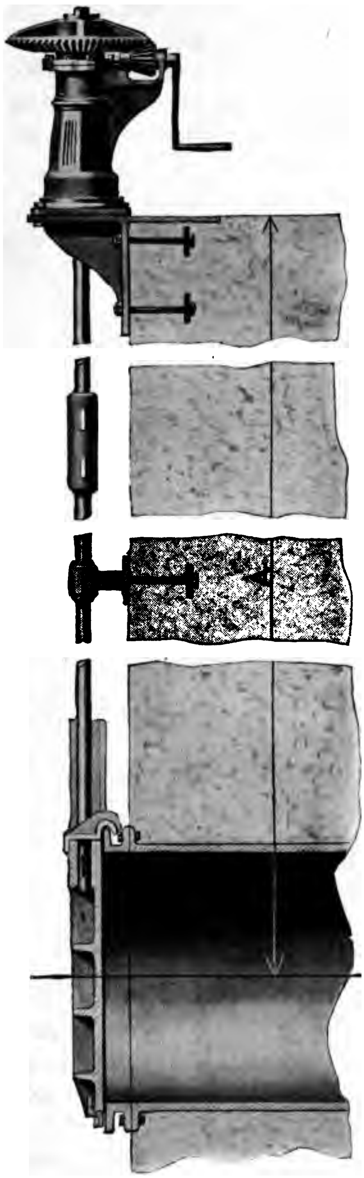


FIG. 119.—Sluice-gate.

The valve is set in position, and also worked at times, by means of a force-pump, to keep it from sticking. All the working parts and bearings of the apparatus are of gun-metal.

An equilibrium relief-valve is placed on the upper side of the gate. This valve is kept closed against the normal pressure of the main by a powerful helical spring, loaded with a weight. Under ordinary conditions, the valve is prevented from lifting by the inertia of the weight and the stiffness of the spring. Any sudden increase of pressure in the main lifts the valve and compresses the spring, the inertia of which is slight. A little water is thus permitted to escape and relieves the pipe from concussion. The relief-valve is provided with an upper and a lower disc, both attached to the same spindle. The action of the valve depends upon the difference of the areas of the two discs. By varying this difference, the valve can be made very sensitive under high pressures. When the valve opens, it affords quick relief, as its large upper disc affords a considerable area of discharge even for a slight lift.

168. Sluice-gates are placed in dams, gate-houses, etc., to control the flow of water through the outlet openings. They are made of cast iron or steel, and are either rectangular or circular. Very large gates are made of wood. The gate is raised and lowered by means of a steel or bronze stem with thread, cut of sufficient length to correspond to the height of the gate. This stem passes through a fixed bronze nut, held in position, and supported, by a cast iron pedestal, operated by suitable gearing (Fig. 119) or by a hand-wheel (Fig. 120). For exceedingly heavy loads, multiple gearing is used, either for hand or motor power. The pedestals are fitted with ball or roller bearings (Fig. 121) to reduce

FIG. 120. — Hand-wheel for Light Sluice-gates.

FIG. 121. — Pedestal with Roller Bearing.

friction. Wide gates are usually fitted with two stems with threads cut right and left hand and the gearing connected with cross-shafts, so as to keep the gate always in true horizontal position. The pedestal may be placed on a bracket or on masonry.

Fig. 122 shows a standard square sluice-gate with a heavy body, reinforced with deep vertical- and cross-ribs. The frame of the gate is of channel form, and has a flange at the top and bottom. The bottom flange is connected to the masonry or to a cast thimble, which may be built in concrete and forms an anchor for the gate. The upper part of the flange of the gate-frame receives the right and left guides, and its bearing-face is fitted with bronze around the gate opening.

Fig. 123 shows a circular sluice-gate with flanges to connect to the masonry or a cast thimble. It is also made with a spigot-end to connect to a standard cast-iron pipe.

The bearing-face of the gate proper is bronze mounted, to correspond to the frame, and is machined true to form a water-tight joint. The right and left side of the gate has a heavy chatter strip that fits close in the grooves formed by the guides and face of the frame, so that the gate moves in machined grooves, and is kept close to the bearing face of the frame, thus preventing chattering, so common in the old style of gates. On the rib side of the gate, bronze adjusting wedges are placed, and the bearings are machined to receive the wedges after they are

FIG. 122.—Standard Rectangular Sluice-gate.

FIG. 123.—Circular Sluice-gate.

finished. They fit in close grooves, and are adjusted by bronze bolts and nuts. A heavy hub is cast in the upper end of the gate in which the stem is set, and is held in position by a flat key passing through the stem and hub. The key is kept in place by split pins, and by this arrangement it is easy to remove the stem, if it becomes necessary.

Many improvements have been made during the past twenty years in the construction of sluice-gates and hoisting apparatus. Heavy gates under heavy pressure are operated by hydraulic cylinders or by electric motors.*

* Figs. 119–123 show the sluice-gates and hoists manufactured by the Coldwell-Wilcox Co., Newburgh, N. Y. Good sluice-gates are, also, manufactured by the Chapman Valve Mfg. Co.; the Ludlow Valve Mfg. Co.; the Coffin Valve Co., etc. The Chapman Valve Mfg. Co. has made sluice-gates as large as 9 by 12 ft.

169. Check-valves are placed in a pipe-line — usually near a reservoir, stand-pipe or pumping station — to prevent the water from flowing back, in case a break should occur in the pipe-line; or from exerting an excessive back pressure on the pumps. The valve consists of a single or double pivoted flap-valve, which is placed horizontally, vertically or

FIG. 124. — Eddy Check-valve.

inclined, in a suitable body casting. Small check-valves are made entirely of bronze. In large valves, the bodies, gates and covers are made of cast iron, and the gate and case-rings, gate-stud and nuts, are of bronze. Figs. 124 and 125 show two kinds of check-valves manufactured by the Eddy Valve Co., Waterford, N. Y.

FIG. 125. — Eddy Check-valve.

If large check-valves are not properly designed, the weight of the gate causes an additional load for the pumps to lift. The weight of the gate may be balanced by extending the shaft to which it is hinged, through the valve body, and attaching to it a lever with a balancing weight, or it may be balanced by the way the gate is hung.

Fig. 126 shows an improved balanced check-valve manufactured by the Rensselaer Valve Co., Troy, N. Y. In this valve the seat is placed vertical to the flow of the water, and the entire weight of the gate is balanced or hung upon long solid bronze hinges, so that the gate hangs lightly against its seat, when no water is flowing. The hinges are connected below the center of the gate and so far within the center of gravity, that the gate is entirely balanced upon the pivot or bearing to which the hinges of the gate are attached. The hinges pass loosely through a pocket near the top of the gate, which is arranged to prevent their balancing or tilting too far either way.

In all check-valves the gate can be removed through a hand-hole or a man-hole. Valves, 14 ins. or more in diameter, are usually provided with a by-pass with a gate-valve (Fig. 125)

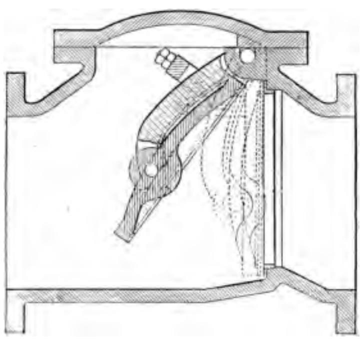


Fig. 126. — Rensselaer Check-valve.

Fig. 127. — Ludlow Pressure-relief Valve.*

170. Pressure-relief valves (Fig. 127) are used as safety-valves on water pipes subjected to heavy stresses from water-hammer. The valve is held in place by a spring protected by a cast iron jacket. By means of a hand-wheel, the pressure on the spring can be regulated. For greater pressures, the valve opens automatically, and allows the water to escape.

171. Air-valves must be placed at summits of pipe-lines, to admit air when the water is drawn off, so as to avoid a vacuum in the pipe, and, also, to permit air that is entrained by the water to escape. These valves should work automatically.

Fig. 128 shows a valve, manufactured by the Eddy Valve Co., for discharging air that collects under pressure at summits of pipe-lines. It is attached to a water-main by a wrought iron pipe, saddle or flange. In this arrangement, the air-valve is kept closed by means of a copper

* Manufactured by the Ludlow Valve Mfg. Co., Troy, N. Y.

float, acting on a lever, but as soon as the water level in the pipe sinks, the float is lowered and opens the valve automatically. The body of the valve is iron, bronze mounted. The float should be made of seamless copper, to make it impossible for water to get into it.

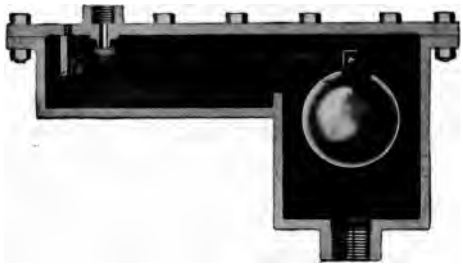


FIG. 128. — Eddy Lever and Float Air-valve.

172. *Automatic poppet air-valves* (Fig. 129) are used at summits of pipe-lines to let the air escape as the pipe is filled with water. The valve is drawn from its seat by the vacuum caused by emptying the pipe. Air is thus admitted into the pipe-line until the valve is again floated to its seat, as the water fills the pipe again. These valves do not discharge air that collects under pressure. Fig. 129 shows an automatic poppet air-valve manufactured by the Ludlow Valve Manufacturing Co.

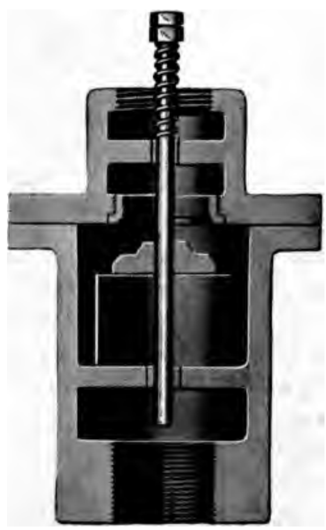


FIG. 129. — Ludlow Automatic Poppet Air-valve.

173. *Automatic Cluster Air-valves.* — For large mains, it is important to provide a sufficient area in the air-valves to prevent partial vacuum in the pipes, when the water is drawn off rapidly. This may be accomplished by using a cluster of air-valves,* as was done in a 38-in. steel main at Rochester, N. Y. (p. 329).

174. Pressure-regulating valves are installed in a pipe-line, or, preferably, on by-passes connected with the pipe-line, at places where it may be necessary to reduce the pressure in the mains. These valves all work on the principle of *throttling* the water — which means reducing the area of the opening through which it must pass. This increases the loss by fric-

* Manufactured by the Rensselaer Valve Co., Troy, N. Y.

tion, and, consequently, causes a reduction of pressure at the outlet side of the valve.

A. O. Doane, who designed some of the pressure-regulating valves installed by the Metropolitan Water and Sewerage Board, of Boston, Mass., gives the conditions which such valves should comply with as follows: * The valves should be simple in design, strong, durable, and reliable, and made of non-corrodible materials. They should be balanced, and not liable to stick to their seats. The valves, which work only in one direction, should be operated from the low-pressure side, and should open and close slowly, so as to avoid water-hammer and surging.

Good pressure regulating valves will reduce pressures as high as 300 lbs. per sq. in. to a pressure of 10 to 100 lbs. With some regulating valves, chattering occurs as the main valve approaches its seat, and

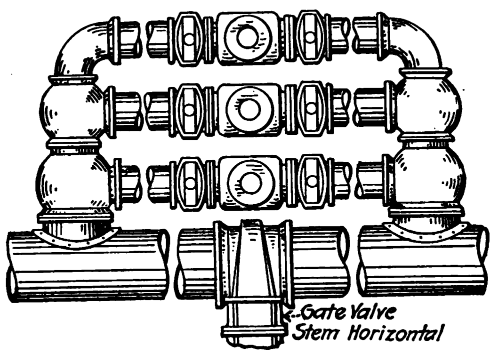


FIG. 130. — Battery of Three Ross Pressure-regulating Valves at Worcester, Mass.

the piston vacillates up and down from the pressure for which it was set. Good valves are free from these defects.

If the pressure-regulating valve is placed in the main pipe-line — as is frequently done — a gate-valve should be provided on each side of it, and a by-pass should be connected to the main pipe-line, so that the regulating-valve can be shut off from the pipe system for inspection or repairs. A better plan is to place the valve in the by-pass. Each valve should be set in a suitable masonry chamber, properly covered.

It is not advisable to use too large a size of pressure-regulating valve, as it will open only slightly and may cause water-hammer. For this reason, it is preferable to use comparatively small valves. In order to obtain the necessary delivery, several of these valves may be placed in battery (Fig. 130).

Pressure-regulating valves should never be placed near summits of

* Jour. New England Water Works Assoc., 1906, XX, p. 1-16.

pipe-lines, as the air entrained in the pipes, which collects at the summits, interferes with the operation of the valve.

Descriptions of some of the principal pressure-regulating valves manufactured in the United States are given below.

175. The Ross pressure-regulating valve * is made in sizes of 3 to 24 ins. in diameter, weighing respectively 180 and 4050 lbs. Fig. 131 shows the manner in which this valve operates. A central stem, carrying two pistons *E*, *F* and the disc valve *G*, is placed in a suitable casting, provided with a top and a bottom cover. The pistons *F* and *G* have the same area, and form a balanced valve. The disc *G* is seated by moving upwards, and is sealed by the action of a leather collar. The piston *E*

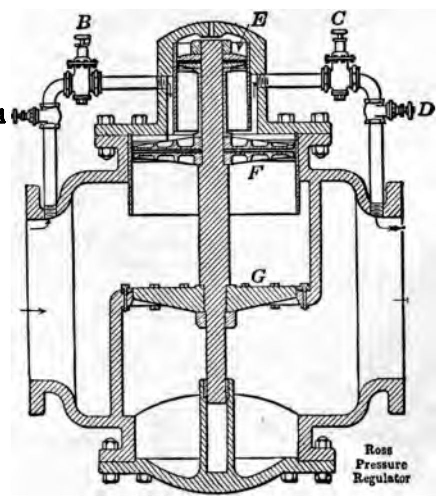


FIG. 131.—Ross Pressure-regulator.

moves in a small chamber in the top cover and, owing to a small vent hole made in this cover, the upper side of piston *E* is subjected only to atmospheric pressure.

The space between pistons *E* and *F* forms the controlling chamber in which the motion of the balanced valve *FG* is regulated. By means of small pipes, each provided with a gate-valve, the controlling chamber is connected both with the inlet (high pressure) and outlet (low pressure) sides of the main valve. The pressure in the controlling chamber is regulated by a small pressure-reducing valve *B* on the inlet pipe, and a relief-valve *C* on the outlet pipe.

The regulator is adjusted by turning the hand wheels of valves *B* and *C*, which changes the pressure at which the regulator will open and

* Manufactured by the Ross Valve Co., Troy, N. Y.

close. They are usually set so that the regulating valve *B* will close, as nearly as possible, at the same pressure which causes the relief-valve *C* to open.

Once properly adjusted, the Ross valve will give close regulation and will work well for a long time. Five 16-in. Ross regulating valves which were installed by the East Jersey Water Company in 1892 are still in service. This valve has been used successfully in the high-pressure fire systems of New York and Brooklyn, and in many other cities.

176. The Mueller pressure-regulating valve * (Fig. 132) has a cast iron body which is divided by a partition wall into an inlet- and an out-

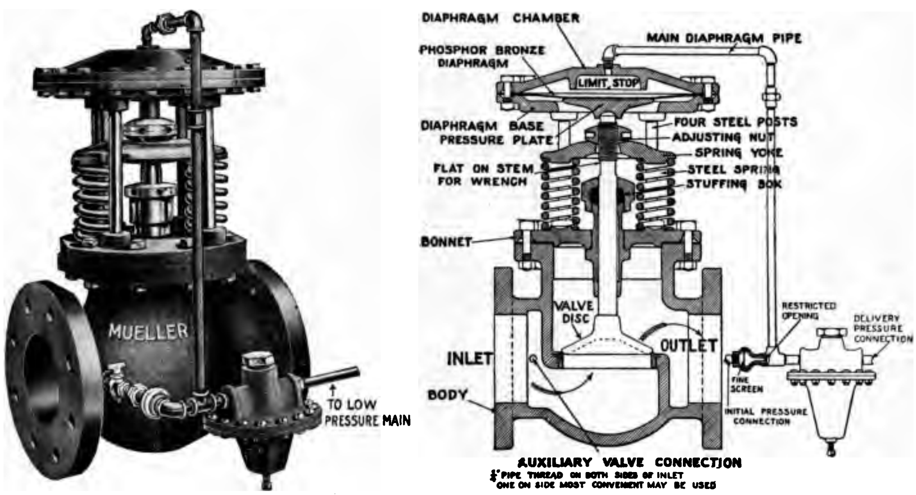


FIG. 132. — Mueller Pressure-regulating Valve.

let-chamber. A circular opening, controlled by a disc-valve, is provided in the partition wall. The stem of this valve passes through a stuffing-box in the bonnet of the body, and extends to a pressure-plate bearing against a phosphor-bronze diaphragm, which is placed in a chamber attached to the bonnet by means of four steel bolts.

Two steel springs are placed on top of the bonnet and bear against a yoke which is connected to the valve-stem of the disc-valve. The height of the yoke above the bonnet can be adjusted by turning a nut on the valve-stem, and the upward pressure exerted by the springs can thus be regulated. By means of a small pipe, connected to the inlet-chamber and to the top of the bonnet, the pressure of the inlet side of the regulator is communicated to the top of the diaphragm.

* Manufactured by H. Mueller Mfg. Co., Decatur, Ill.

The disc-valve is controlled entirely by a small auxiliary valve, which is placed on one side of the body of the regulator. This valve is a small pressure-regulator, which is connected to the inlet side of the large regulator and, also, to the low-pressure pipe. The main valve is controlled entirely by the auxiliary valve, which closes and opens the main valve by admitting initial pressure to, and releasing it from, the diaphragm-chamber, according to the demands of the delivery side.

The reduction of pressure between the inlet and outlet of the main regulator depends upon the height to which the main valve is raised from

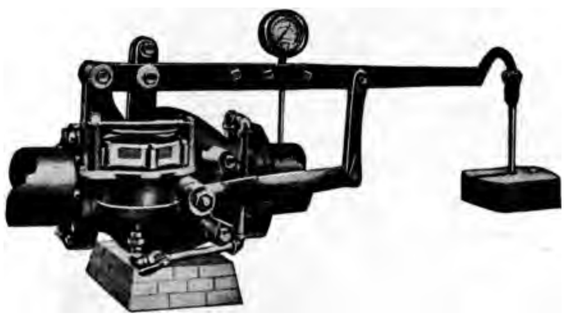


FIG. 133. — Union Pressure-regulator.

its seat. This height is regulated by varying the pressure exerted by the springs.

The diaphragm-chamber is connected to the full size low-pressure main, a short distance from the valve, by a pipe having the same size as the outlet connection of the auxiliary valve.

177. The Union pressure-regulator * (Fig. 133) is controlled by the pressure of the water at the outlet side of the valve, acting under a diaphragm. A lever and weight, acting on the upper side of the diaphragm, balance the pressure on its under side, when the desired point is reached. An interior conical valve with four ports, which connect the inlet with the outlet, is rotated by means of the lever-arm which is attached to its stem. The ports are opened or closed as the pressure on the diaphragm diminishes or increases. The regulator is adjusted for various pressures by varying the weight on the lever. The more weight is added, the greater will be the pressure on the outlet side. A vent on the upper side of the diaphragm insures its operating only against atmospheric pressure. The water acting on the bottom of the piston tends to close the regulator. The opening of the main valve is controlled by the weights at the end of the lever.

* Manufactured by the Union Water Meter Co., Worcester, Mass.

178. Foster Pressure-regulator* (Fig. 134). — In this regulator, the pressure which causes the main valve to open is produced by a spring, placed in a chamber above a piston follower, and brought to the desired amount of pressure by means of an adjusting screw. By turning this screw to the right or the left, the distance of the main valve from its

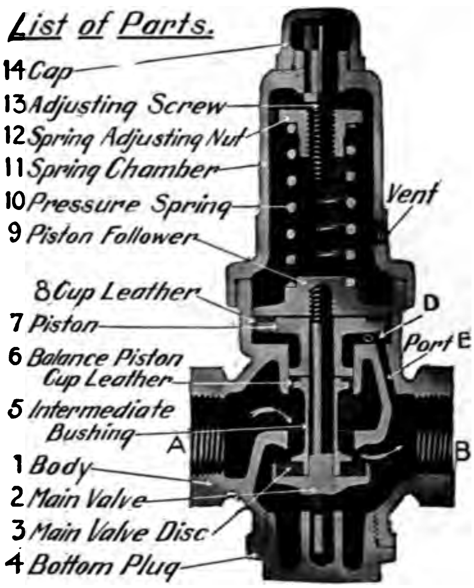


FIG. 134. — Foster Pressure-regulator.

seat — and consequently the area through which the water can flow — is regulated. A vent-hole in the spring chamber insures that the top of the piston is only subjected to atmospheric pressure and, also, serves as a tell-tale, when the packing needs renewal. At the outlet opening of the regulator, there is a port which admits the water under the piston. If the pressure of the water is greater than the pressure of the spring, it will force the piston upwards and close the main valve partly or entirely. The regulator maintains a constant delivery pressure regardless of any variation in the initial or supply pressure at the inlet side.

The main valve is provided with a soft disc, which is easily renewed. The regulator is made in sizes of ½ to 12 ins. For larger sizes up to 20 ins., the regulator is constructed as shown in Fig. 135.

179. Golden-Anderson Pressure-regulator† (Fig. 136).—The body is made of cast iron and the upper portion is lined with bronze. The

* Manufactured by the Foster Engineering Co., Newark, N. J.

† Manufactured by the Golden-Anderson Valve Specialty Co., Pittsburgh, Pa.

piston valve *B*, which is of bronze, is fitted with rubber or leather cups and discs, in order to prevent the metal parts from coming in contact, when the valve is opened or closed.

The valve is operated by hand, or by electricity, by means of the auxiliary valve *K*. When this valve is opened, the water is admitted,

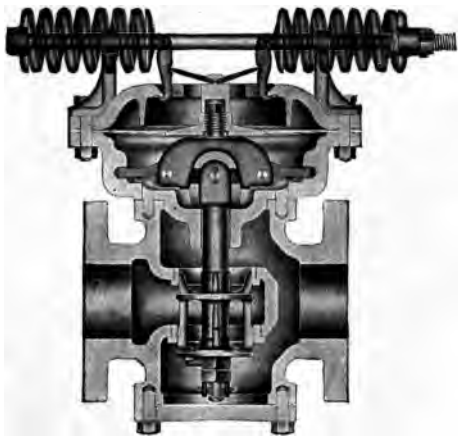


FIG. 135.—Foster Pressure-regulator for Large Mains.

through the ports *L* and *M*, on top of piston-valve *B*, which is forced down to its lower seat, as the area of the top of this valve is larger than that of its bottom area, and the valve is thus closed. When the valve is to be opened, the auxiliary valve *K* is closed. This permits the water on top of the piston-valve *B* to flow out through ports *M* and *N*, as the pressure under the piston-valve forces it to its upper seat.

For *fire service*, the valve is arranged so that it can be operated by hand, or by electricity from a distance. When the valve is operated electrically, the current is turned into the electric solenoid *E*, by pressing a push-button or throwing a switch in and out of contact. The solenoid causes a cam *H* to operate the valve-lever, thus opening the high-pressure valve, allowing the water to flow through ports *L* and *M* on top of piston-valve, which is thus pushed to the lower seat. In order to open the valve, the button is pressed or the switch is thrown in and out of contact, which causes the cam to operate, permitting the valve-lever to return to its regular position.

The electric solenoid is of the plain magnet type, of heavy and substantial construction, and can be furnished for direct or alternating current of the various voltages. There is no waste of electric current; also no possibility of the solenoid's burning out, so that economic and uninterrupted service is assured at all times. The valve is provided with hand-stop-attachment for permanent closure.

180. Golden-Anderson Altitude Valve (Fig. 136*a*) is an arrangement of the valve described above, which is used in connection with tanks, stand-pipes, and reservoirs. It is opened and closed automatically by

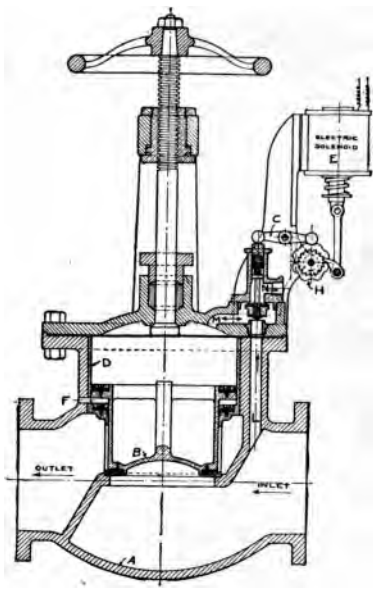


FIG. 136. — Golden-Anderson Pressure-regulator.

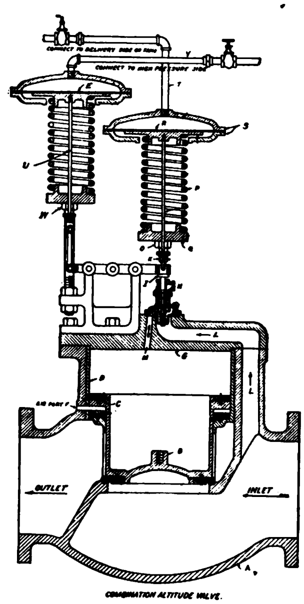


FIG. 136*a*. — Golden-Anderson Valve for Altitude or Fire Service.

the water pressure from the reservoir which acts through pipe *T* on diaphragm *R*. When the water surface in the reservoir sinks below a desired level, the pressure on top of diaphragm *R* is reduced, and the spring beneath it moves the diaphragm, and the spindle *K* attached to it, upwards, thereby permitting the water above the piston-valve *B* to escape, as the water from the supply pipe forces valve *B* open. This valve can, also, be arranged to close and open electrically, as described above.

For fire service, the valve can be arranged to close automatically as the pressure is increased, regardless of the height of water in the reservoir, so as to insure immediately an increase of pressure at the desired point, without waiting until the reservoir has become full. In this case, the valve is provided with two diaphragms acting against springs (Fig. 137), one of them being connected with the delivery side while the other is connected with the high-pressure side.

180*a*. Valve-boxes, called also roadway-boxes, are placed over valves set in roadways or sidewalks, to protect them, and to make them accessible.

Fig. 137 shows a common type of valve-box for a small gate-valve. It consists of three castings: a base, a shaft, which may be made of different lengths, as required, and a top provided with a cover. The top and shaft are united by a screw joint, which makes it possible to adjust the top of the box to the street level. In Fig. 137*a*,* the two castings forming the valve-box have simply a telescopic joint, which permits the moving of

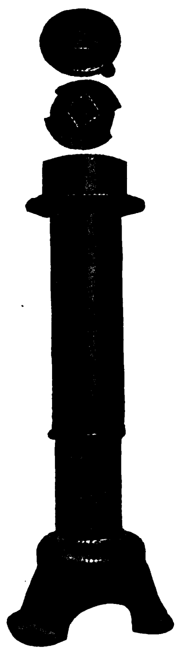


Fig. 137.

Fig. 137*a*.

Valve-boxes.

the upper section up or down, without disturbing the lower section. This feature is advantageous because it practically eliminates broken joints from the crushing down of the box by road rollers or other heavy traffic, and also allows quick readjustment, in event of frost upheavals, or changes in grade. The inside and outside covers lock like breech-blocks, as lugs wedge under the shoulders on shoulders on the inner sur-

* The valve-boxes shown in Figs. 137*a* and 137*c* are manufactured by the S. E. T. Valve and Hydrant Co., New York.

face of the box, when the covers are turned. Plug-covers are also frequently used.

The gate-boxes described above, which are suitable for valves 2 to 8 ins. in diameter, rest on the tamped refilling of the trench. For large

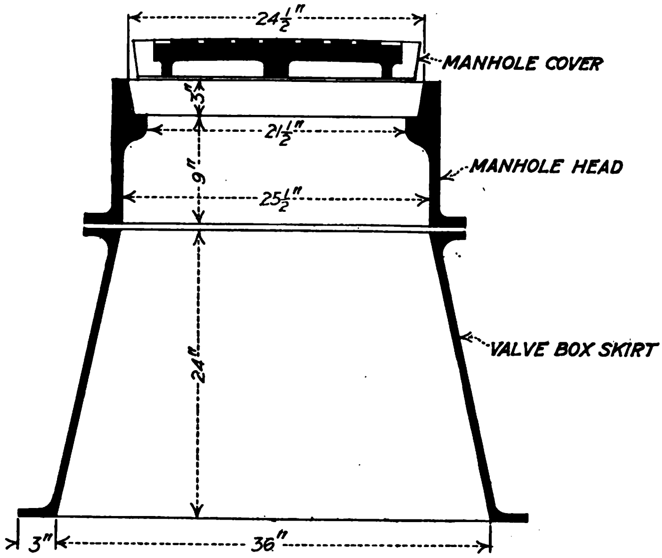


FIG. 137b.—Standard Valve-box, New York City.

FIG. 137c.—Sectional Valve-boxes.

valves the gate-box may consist of a conical casting, provided on the top and the bottom with a flange, and surmounted by a manhole-head with a cover. (Fig. 137b.)

Fig. 137c shows two types of a valve-box with an adjustable manhole-top which forms a very good housing for a gate-valve, etc. It is made cylindrical or rectangular, and consists of sections of cast iron which are put together without the use of any bolts. The cylindrical box is provided with an adjustable top, which has a range of $5\frac{1}{4}$ ins. It has an inside diameter of 24 or 36 ins. The rectangular box has inside dimensions of 15 by 17 ins.

The adjustable top of the cylindrical housing — which may also be placed on a masonry manhole — consists of a frame made in two parts, the upper one riding on the lower one. It is provided with a corrugated cover, which is placed in the upper part. The lower half is fitted on the inside with a triple set of inclined planes, the upper half being provided with corresponding planes so that when the upper half is placed in position, its planes ride on those of the lower half. Rattling or dishing out of the cover is prevented by a deep flange on the under side of the cover which extends down below the frame-lid. The upper half of the frame, in which the cover rests, is provided with a rim, 4 ins. wide. This prevents the cover-frame from being disturbed by sudden shocks from heavy or swift traffic. In ordinary manhole-covers the width of rim is only from 1 to $1\frac{1}{2}$ ins.

CHAPTER XII

HYDRANTS

181. Hydrants, connected by means of branch-pipes with the water mains, are placed at convenient points in the streets of a city or town — usually at the street corners — to make it possible to draw water for extinguishing fires, and for street sprinkling, etc.

Two types of hydrants are in use, *viz.: post-hydrants* and *flush-hydrants.* The former project 2 or 3 ft. above the side-walk, while the latter are placed in suitable vaults or boxes below the surface of the side-walk or of the roadway. Post-hydrants are employed almost universally in America, on account of the ease with which they can be found, in case of fire. The use of flush-hydrants is confined to a few large cities, especially for congested districts, where the side-walks must be kept unobstructed.

In order to furnish an ample supply of water for extinguishing fires, the area of the stand-pipe of a hydrant, called the barrel, should be about 20 to 40 per cent greater than the areas of all its nozzles, more allowance being made for large hydrants than for small ones. All turns or corners in the barrel must be rounded off, so as to reduce the head lost by friction to a minimum. There is considerable difference in hydrants in this respect.*

182. *Valves.* — Two kinds of valves for controlling the flow of water into the hydrant-barrel are in common use, *viz.:* plug-valves and gate-valves. The former consists generally of a number of leather discs, attached to the valve-rod at its lower end, and kept in place by nuts and washers; or, it may be a conical metal valve, which is faced in some designs with leather or rubber. This valve is so placed that it is forced, when closed, against its seat, by the pressure in the main, and it is called, therefore, a compression-valve. The other kind of hydrant-valve consists of some type of gate, bearing against a faced-seat, and moved up and down vertically by its *rod* or *stem.*

The valve-stem passes through a stuffing-box, made either of bronze or lined with bronze, which is provided in the top of the hydrant, and ends in a five-sided nut which can be turned to open or close the main valve by means of a special wrench, made to fit the nut. An arrow

* "Experiments on Hydrants," by Charles L. Newcomb in Trans. Am. Soc. M. E., 1899, XX, p. 494.

should be cast on the top of the hydrant, to indicate which way to turn the nut in order to open the hydrant. The stem is usually made of wrought iron or steel, and is covered at its upper end by a bronze sleeve of sufficient length.

183. *Drip-valves.* — When the valve of a hydrant is closed, after the hydrant has been in service, the barrel remains full of water, and must be drained, especially in winter, to prevent the water in the barrel from freezing and rupturing the barrel. All hydrants are provided for this purpose with a *drip- or waste-valve.* In sandy and gravelly soil, the waste water is usually discharged into the ground, which quickly drains the hydrant. This cannot, however, be done when the soil consists of clay. In this case, the waste water must either be discharged through a drain-pipe into some nearby sewer; or enough broken stone must be piled around the bottom of the hydrant, to provide sufficient void space between the stones for draining the hydrant. If the bottom of the hydrant should be below the level of the ground water, the barrel will have to be pumped out after being used. A frost-casing is often placed around the lower part of the hydrant barrel, as an extra protection in winter, but has been found to be of little advantage.

184. *Nozzles.* — Hydrants are made with one to four nozzles, which have usually inner diameters of either $2\frac{1}{2}$ ins. to fit the standard fire-hose of this size, or of $4\frac{1}{2}$ ins., for the 4-in. suction hose of steam fire-engines.

185. The nozzles are generally closed by screwing on caps which are fastened to the barrel by chains, to prevent their being mislaid. In hydrants subjected to great pressure, however, a special stop-gate is provided for each nozzle.

186. Setting of Hydrants. — The hydrant is placed on a flat stone or block of concrete, and is usually connected with the street main by a 4-in. or 6-in. branch-pipe, according to the number of nozzles that are to be supplied. A gate-valve, protected by a suitable gate-box, should be placed on the branch-pipe, for shutting off the water from the hydrant, when repairs are required.

187. Standardizing Hydrants. — It is important to standardize hydrants as much as possible. Fire companies which have been brought from neighboring towns, to assist in putting out a great conflagration, have sometimes been unable to render assistance, because they could not couple their hose to the fire nozzles, on account of a difference in the pitch of the thread. For some time, committees of the leading water works associations of America have been occupied in drawing up standard specifications of hydrants. At the Annual Convention of the American Water Works Association, held in 1916, the standard specifications for hydrants, given in the Appendix, were adopted.

Large cities have generally standard hydrants of their own, but the majority of water works are supplied with hydrants by manufacturers who have made a specialty of such appurtenances. In order to illustrate the details that enter into the construction of hydrants, we give below descriptions of some of the leading hydrants which are now in the market in America. Good hydrants, not described in this book, are also made by: The Kennedy Valve Mfg. Co., Elmira, N. Y.; Pratt & Cady, Hartford, Conn.; The Bourbon Copper and Brass Works Co., Cincinnati, Ohio, etc.

188. The Mathews fire hydrant* (Fig. 138) consists of a cast iron stand-pipe, curved base, and hydrant head, in which the working parts are placed. The stand-pipe is provided with a screw-thread at its lower end, and is screwed into the hydrant base. An outside casing, the upper end of which makes a telescopic joint with the body or post of the hydrant, adds finish and strength to the hydrant, and provides an air space around the stand-pipe, which prevents the formation of ice in the hydrant. This case has an end play, or vertical motion, of several inches, independently of the hydrant proper, and accommodates itself to the upheaval of the ground by frost, thus preventing the heaving and breaking of the hydrant or foot bend.

The sliding case has another advantage. It makes it possible to unscrew the stand-pipe from its foot-piece, and to raise it, with the working parts it encloses, to the surface of the street, leaving the outer casing behind, to keep the earth around the hydrant in place.

The main valve is of the compression type. It is of leather and is turned to a conical shape, to fit the beveled valve-seat. This valve opens downward, and the water pressure helps, therefore, in closing the valve and holding it in place. If the hydrant should be broken, the streets will not be flooded, as the main valve will remain closed.

The main valve is attached to a rod or spindle, which has a screw-thread, cut at its upper end. This rod passes through a stuffing-box, and its screw-end engages with a revolving pentagonal nut at the top of the hydrant. By turning this nut to the right, the main valve is opened, and by reversing the motion, it is closed.

The drip-valve is attached to the rod of the main valve, and acts, therefore, automatically, closing when the main valve is opened, and vice versa. The guides of the drip-valve are closely fitted to grooves in the lower end of the stand-pipe. This prevents trembling or vibration, in opening or closing the hydrant under pressure, the rod and main valve being held rigidly to the center of the hydrant.

The hydrant is screwed tight into its base by the manufacturers.

* Manufactured by R. D. Wood & Co., Philadelphia, Pa.

When it is to be taken up for repairs, a chain or stout rope is placed around the body of the hydrant, immediately below the nozzle, through which a couple of levers, 6 or 8 ft. long, are passed. Two men can usually unscrew the hydrant from its base by means of the levers and take it up. The hydrant can then be easily repaired, if necessary.

FIG. 138.—Mathews Hydrant.

FIG. 139.—Mathews Double-valve Hydrant.

189. Mathews double-valve fire hydrant (Fig. 139) provides double security against breakage, by the use of two main valves, one above the other. The lower valve acts as an auxiliary to the upper valve, and makes it possible to take up the hydrant, without shutting off the water. The lower valve is held tight against its seat by the water pressure and a spring. When the upper valve is lowered to open the hydrant, it forces the lower valve, also, down.

When required, each of the nozzles of this hydrant is provided with an independent cut-off valve.

190. The Eddy fire hydrant* (Fig. 140) has a conical, rubber-faced valve, which is raised or lowered by turning a screw-threaded bronze valve-stem. This stem passes at the top of the hydrant through

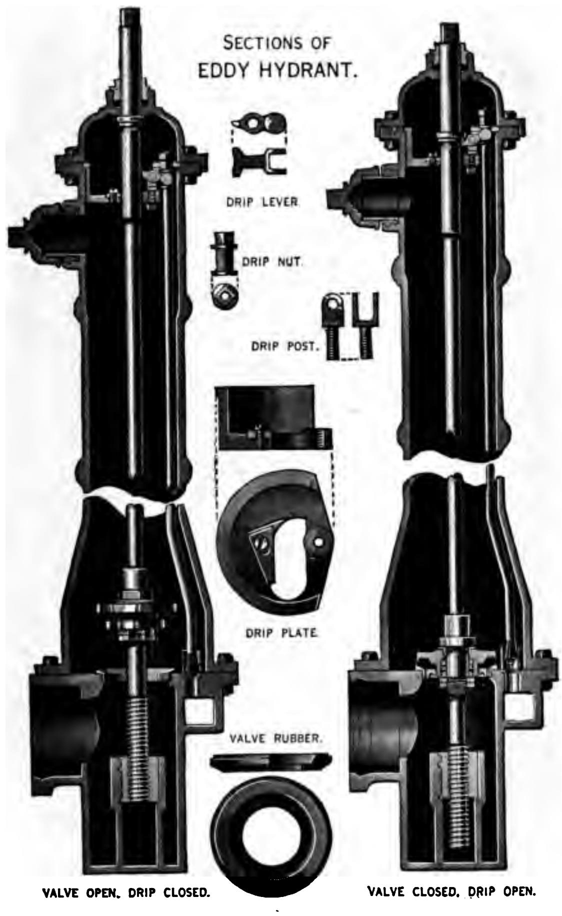


FIG. 140.—Eddy Fire Hydrant.

a stuffing-box, and screws at the bottom of the hydrant barrel into a nut in a post. The valve is drawn to its seat by the screw-end of the stem and remains closed, even if the barrel of the hydrant should be broken. The valve has below its conical part a straight cylinder piece, which enters the valve opening and shuts off the water gradually, so that only

* Manufactured by the Eddy Valve Co., Waterford, N. Y.

a trifling flow remains, when the valve reaches its seat. This arrangement prevents water-hammer. The rubber-face is attached to the valve, so that it can easily be replaced and the valve, being free to revolve on its stem, adjusts itself readily to its seat.

The hydrant barrel is enlarged at the valve-seat, so as to provide an ample water passage around the valve. The manufacturers state that experiments with pressure gauges have shown that there is only a loss of less than two per cent of pressure between the inlet and nozzle of the hydrant.

A drip-plug, placed at the side of the valve-seat, drains the hydrant barrel through apertures at the side of the cup. The outflow from the cup is controlled by a solid, tapering rubber drip-plug, which is screwed to the end of the drip-rod and further held in place by a brass nut. This rod with the drip-plug is raised or lowered by a small drip-lever, pivoted to a post in the drip-support near the top of the hydrant. The lever has a forked end which moves the drip-rod, and its other end engages with the bronze covered valve-stem on which a groove and a collar are provided. When the valve is closed, the drip-cup is wide open, and one end of the lever is in the groove on the stem. As the valve is being raised, the end of the drip-lever at the stem is shoved upwards and the cup is closed. When the valve is shut down, the collar on its stem forces down the end of the drip-lever and thus raises the drip-rod and plug, and opens the apertures in the cup. Less than one revolution of the valve-stem suffices to open or close the openings in the drip-cup. The drip arrangement is, therefore, controlled automatically and positively. The drip-plug and rod can be removed, while the pressure is on the valve, and can be adjusted from the top, without being taken out of the barrel.

As the valve-stem rises or lowers with the valve, its upper end serves as an indicator to show the position of the valve. The hydrant is thoroughly bronze-mounted and its working parts can be easily removed from the barrel, if it should be desired to make the hydrant serve as a blow-off for flushing the water main.

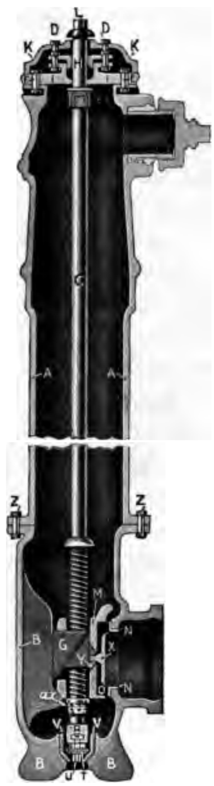
191. The Ludlow Slide-gate Fire Hydrant* (Fig. 141). — The gate is made of cast iron and is faced with a heavy, solid, rubber gasket, which is held in the gate by a bronze plate. The seat-ring, against which the gate closes, is made of bronze. On the back of the gate, there is an incline with chilled face on which a solid bronze wedge-nut moves, in opening and closing the gate. The gate-stem passes through two guide holes in the back of the gate, and screws through the bronze wedge-nut. A bronze sleeve passes through a stuffing-box, at the top of the hydrant, and the stem is held at the bottom in a bronze drip-cup. A collar on

* Manufactured by the Ludlow Mfg. Co., Troy, N. Y.

the sleeve, just below the hydrant-cover, prevents the stem from rising, as it is turned. The upper part of the stem is made of wrought iron, but the lower, threaded part is made of bronze. The sleeve which passes through the stuffing-box is also of bronze.

When the hydrant is being shut off by revolving the gate-stem in the proper direction, the gate moves downward until it strikes a stop at the

A — Barrel or Stand Pipe.
B — Bottom.
C — Stem.
D — Follower or Dome Bolts.
E — Packing Plate Bolts.
G — Bronze Wedge Nut.
H — Bronze Sleeve.
I — Packing Plate.
J — Packing Gland or Follower.
K — Dome or Cover.
L — Top Nut.
M — Gate.
N — Bronze Seat Ring.



O — Gate Rubber.
P — Nozzle Cap.
Q — Bronze Nozzle.
R — Bronze Screwed End Drip Piece.
S — Bronze Drip Washer.
T — Drip Rubber.
U — Bronze Drip Nut.
V — Bronze Drip Cup.
W — Bronze Drip Bolt.
X — Bronze Gate Plate.
Y — Bronze Gate Plate Nut.
Z — Flange Bolts.
ax — Gate Locking Device.

FIG. 141. — Ludlow Slide-gate Hydrant.

bottom of the projection in the back of the hydrant barrel, which arrests the travel of the gate. The wedge-nut continues its downward travel, and presses against the incline on the back of the gate, which is in this manner wedged firmly against the seat. All grinding against the bronze seat-ring is prevented by projections at the top and bottom of the face of the gate.

A rubber-faced bronze drip-valve screws to the end of the gate-stem. It moves in a corrugated bronze drip-cup, which is set in the bottom of the hydrant. The drip-valve does not open until the gate is closed, and it closes before the gate is released from its seat. This drip is located in the extreme bottom of the hydrant, and drains the entire stand-pipe. The drip-valve never leaves its cup and cannot become clogged, as it revolves with every motion of the stem.

This hydrant is also provided with a gate-locking device, which locks the gate after it is seated and the drip-valve is open. This device prevents flooding of the street, in case the stand-pipe or barrel is broken, and also holds the gate tight against its seat, when it is necessary to remove the packing-plate. All the bearings of the hydrant are fully bronze mounted. The stem with the gate, drip-valve, etc., can easily be taken out of the hydrant by removing the dome and packing plate.

FIG. 142. — Ludlow Hydrant with Balanced Valve.

192. The Ludlow fire hydrant with balanced valve (Fig. 142) is made by the same company that manufactures the hydrant described above. This improved hydrant has at the bottom a double poppet-valve, to control the inlet opening. The balanced valve permits the operating of the fire hydrant with the greatest possible ease under high or low pressure. All the inside working parts can be removed without disturbing the hydrant barrel.

193. The Chapman post fire hydrants * (Fig. 143) has a sliding gate or plug, which shuts off the water gradually from the main, thus preventing all water-hammer. The gate is faced with a heavy renewable seat-ring of special rubber, which is held in place by a plate and nut. When the gate is closed, its rubber face bears against a bronze seat-ring in the hydrant body. The gate is guided in its movement by ribs cast on the body of the hydrant, which engage with grooves in the gate.

The gate is operated from the top of the hydrant post, by means of an iron rod, provided at the top with a bronze nut, and extended by a threaded bronze spindle that works in a solid bronze wedge-nut at the back of the gate. A collar on the gate-stem at the stuffing-box in the cap of the hydrant prevents the stem from moving vertically. In closing the hydrant, the action of the spindle forces the gate downward until it

* Manufactured by the Chapman Valve Mfg. Co., Indian Orchard, Mass.

comes in contact with a stop in the bottom of the hydrant body. As the turning of the gate-stem is continued, the bronze wedge-nut, acting on the inclined surface on the back of the hydrant body, forces the gate horizontally and squarely against its seat. In opening the hydrant, the wedge-nut is released first, and the gate is forced clear of its seat, and is held thus during its movement. At no time, during the opening or closing, is there any frictional contact between the gate and its seat, all

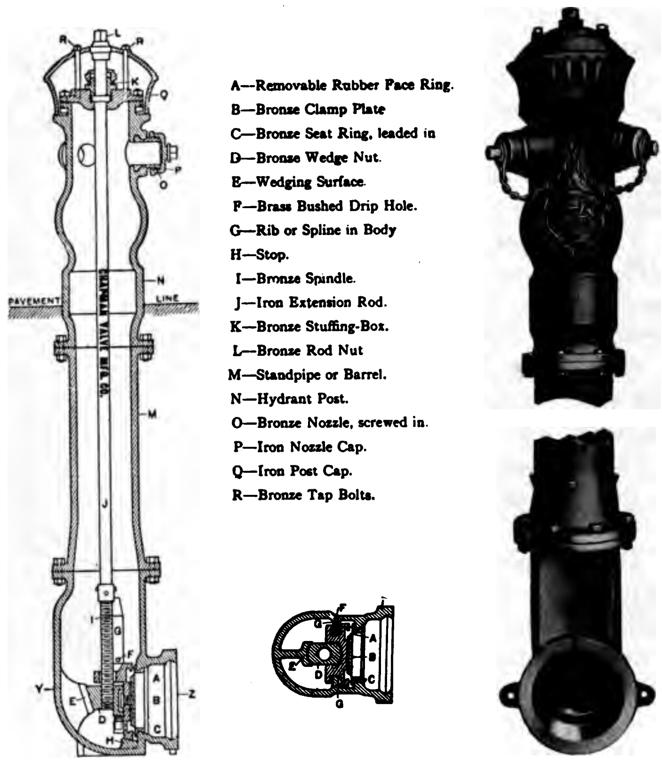


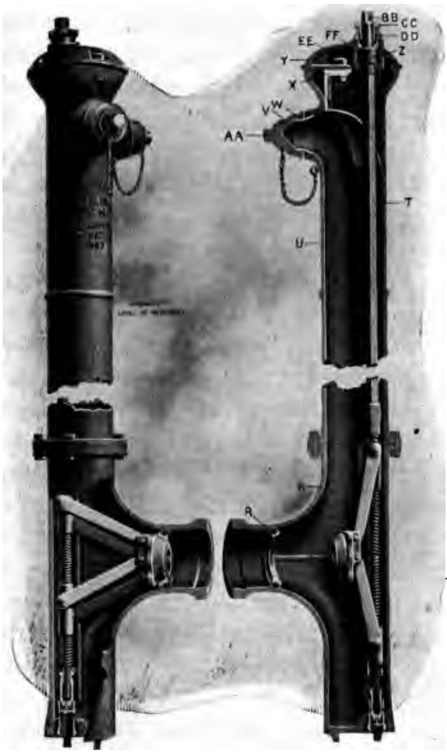
FIG. 143. — Chapman Post Fire Hydrant with Removable Rubber-faced Plug.

wear of the rubber-face being thus avoided. The water passages around the gate are made ample so that the loss of pressure in the hydrant is reduced to a minimum.

The drip-outlet, which is bushed with bronze, is located in the side of the hydrant-body on the level of the water in the main. It is controlled automatically by the motion of the hydrant gate. When the latter is closed, the drip-outlet is open. The moment the gate begins to rise, its drip-outlet is sealed and remains thus until the gate is closed again.

The nozzles are of bronze with male hose-threads, and have iron nozzle caps and chains. The stuffing-box of the gate-stem is of bronze with screw packing-nut, placed in the top of the post, and protected by the post or hydrant cap, which is held in place by two bronze tap-bolts.

The hydrants are furnished with bell, flange or screw-ends, and with or without post eaves, as may be desired.



R — Brass Gate or Seat Ring.
S — Hydrant Bottom.
T — Operating Rod.
U — Hydrant Standpipe.
V — Nozzle Cap.
W — Brass Nozzle (threaded).
X — Deflector.
Y — Hydrant Head.
Z — Hydrant Sleeve.
AA — Nozzle Cap Nut.
BB — Operating Nut.
CC — Brass Follower.
DD — Brass Stuffing Box.
EE — Deflector Bolt.
FF — Sleeve Set Screw.

FIG. 144. — Corey Hydrant.

194. The Corey hydrant* (Fig. 144) consists of a cast iron stand-pipe, bottom, and head, enclosing the interior working parts. The diameter of the stand-pipe is made extra large, in order to reduce the loss of pressure by friction to a minimum. The shape of the hydrant bottom gives an anchorage in the ground below the frost-line that holds it securely, and makes heaving by frost impossible. No frost casing is, therefore, required.

The special feature of this hydrant is the main valve, which is moved horizontally to one side of the hydrant, when it is opened, thus giving a

* Manufactured by the Rensselaer Valve Co., Troy, N. Y.

wide passage for the water. A deflector, bolted to the hydrant-head, turns the stream of water by easy curves into the nozzles.

By the arrangement described above, the hydrant delivers a solid stream of water, equal to the full size of the valve opening, with a minimum loss by friction. The valve consists of an iron casting, upon which a strong, specially prepared rubber ring is attached, by means of a bolt, washer, and nut. This rubber washer can be easily removed and replaced. Four cast iron, brass mounted arms are attached at one end to the valve and at the other to stem-nuts, which are moved up and down on a double threaded stem of hot rolled Tobin bronze, when the operating nut at the upper end of the stem is turned. The arms hold the valve in position, when closed, by the half-ball ends of the arms, running in grooves in the back of the hydrant, and they form four strong, solid braces between the back of the hydrant and the valve. The only stresses coming on the threaded spindle are those caused by moving the half-ball parts of the arms up and down in their grooves, and holding them in position. The knuckle-joint formed by each pair of opposite arms gives great power at the opening or closing of the main valve, and increasing speed as the valve is moved from its seat towards the side of the barrel, and diminishing speed as the valve is returned to its seat. The first turn of the spindle, when the main valve is wide open, moves the valve two inches, while, at the time of closing, each turn of the spindle scarcely moves the valve $\frac{1}{16}$ in. This arrangement reduces the danger of water-hammer to a minimum. The valve is closed very tight, with slight application of power, by means of the operating-lever.

The rubber drip-valve is located below the bottom of the connecting pipe, and drains the main hydrant bottom through a perforated drip-barrel, having a sufficient number of holes at the proper point for perfect drainage.

The drip-cup is fastened to a cylindrical sleeve-nut into which the lower end of the spindle screws. As the spindle is turned to the right or the left, the sleeve-nut with the drip-valve is moved up or down in its barrel, the water pressure forcing the cup against the side of the barrel, thus ensuring water-tightness. When the drip-cup has been raised to the proper level, the hydrant is drained through the perforation in the drip-barrel. As soon as the cup is lowered sufficiently to cover the perforations, the drainage is stopped.

All of the working parts of the hydrant can be easily removed through the top of the hydrant, when the head has been taken off. In case the stand-pipe should be broken off by some accident, the valve will still remain tight, as it is braced by the four arms against its seat.

The Corey hydrant is furnished, when desired, with independent hose-nozzle gates (Figs. 145 and 145*a*), placed inside or outside of the hydrant. This arrangement makes it possible to keep one or more lines of hose constantly attached to the hydrant for quick service at factories, etc.

195. The Smith standard fire hydrant* (Fig. 146) is of the compression type. It consists of a cast iron stand-pipe of large diameter, which is bolted to the shoe and covered by a cast iron bonnet. The principal working parts are of bronze. All of the inner parts of the hydrant can be taken out, by means of special wrenches, through the top of the hydrant, when the bonnet has been removed.

The main valve consists of a bronze upper plate and a cast iron lower plate, between which leather packing-rings are placed. The upper plate

FIG. 145. — Corey Hydrant with Independent Gates for Hose-nozzles.

FIG. 145*a*. — Corey Hydrant with Independent Gates for Hose-nozzles.

FIG. 146. — Smith Low-pressure Hydrant.

is provided with splines (guides) which move in grooves in the valve seat, thus preventing all lateral motion or vibration of the valve, even when the pressure is as high as 100 lbs. per square inch. The rod of the main valve is of steel, but is bushed with bronze where it passes through the packing-box. This rod is attached at the bottom to the main valve, by means of nuts, and is operated by a pentagonal bronze nut at the top. In opening, the main valve moves downward until it reaches a post, cast in the bottom of the shoe. As the valve closes with the pressure, it shuts

* Manufactured by the A. P. Smith Mfg. Co., East Orange, N. J.

off the water automatically, in case the stand-pipe is broken, a pressure of 5 lbs. per square inch being sufficient for this purpose.

The drip-hole, which is lined with lead, is placed low enough to drain the hydrant barrel completely. The bronze drip-cup is operated by a drip-rod, consisting of a wrought-iron pipe which is attached to the main valve rod by a cross-bar, held in place by a set-screw. The drip-rod moves, therefore, always with the main rod, opening the drip-cup when the main valve is closed, and vice versa. Perfect drainage is thus obtained.

FIG. 147. — Smith High-pressure Hydrant.

196. The Smith high-pressure hydrant* (Fig. 147) is of similar construction as the *Standard hydrant*, but is made stronger in all of its parts and has a larger diameter. In this hydrant each nozzle is provided with a special sliding gate.

197. The Coffin compression fire hydrant † (Fig. 148) is very simple in construction. The valve, which is either leather or rubber-faced, is raised vertically from its seat at the bottom of the hydrant by revolving a square or pentagon nut at the top of the hydrant, into which the valve-stem works. The valve closes with the pressure, thus avoiding water-ram. No frost-case is required, as waste-holes near the bottom of the hydrant drain the barrel, when the valve is closed. When the valve is opened, the drain-holes are closed by a leather strip fastened as shown in Fig. 148. The dome of the hydrant can be readily removed without disturbing the stuffing-box packing, and the valve with its stem and the waste-valves can then be easily pulled out of the barrel.

Each standard hydrant is provided with two $2\frac{1}{2}$-in. and one steamer nozzle, $4\frac{1}{2}$ ins. in diameter, which are screwed in and then pinned. The operating parts are of solid bronze, and provision is made for oiling the screws. The stand-pipes are large, and the hydrant has no pockets where mud, etc., can lodge.

* Manufactured by the A. P. Smith Mfg. Co., East Orange, N. J.

† Manufactured by the Coffin Valve Co., Boston, Mass.

The hydrant is drained from two opposite sides, to prevent its being undermined. The drainage valves are operated by the valve-stem, so as to close the drainage openings when the valve is open and to open them when the valve is closed.

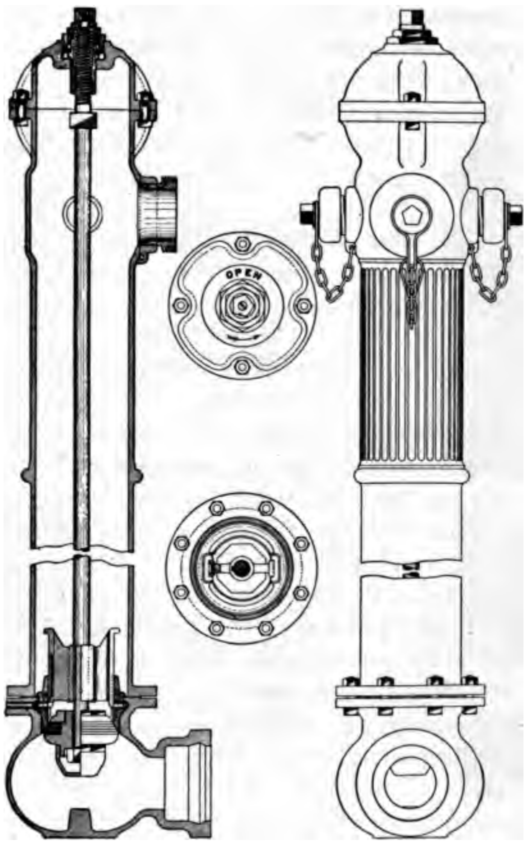


FIG. 148. — Coffin Fire Hydrant.

198. The Holyoke hydrant* (Fig. 149) has a gate-valve which opens downward, giving thus an unobstructed waterway. This valve has a cast iron body with a bronze nut cast into it, where the valve-screw passes through. A bronze drip-plate is cast in the back of the body where it comes in contact with the back shoe. The valve-face is of bronze, and is shrunk onto the iron valve body. The valve-rod fits into sockets at either end, and is so constructed that the valve will remain

* Manufactured by the Norwood Engineering Co., Florence, Mass.

closed, even if the head or part of the hydrant should be broken. The drip device is constructed as follows:

FIG. 149.—Holyoke Fire Hydrant.

The back shoe, or groove-plate, is held in place by the back screw and drip-screw. This plate is of bronze and is planed down to a smooth surface. The drip-valve consists of a plate, held to the main valve by 2 cotter pins, and slides up and down in the groove of the back shoe. The drip drains the hydrant to a point below the main. A drip-box can be attached to the valve-case. It protects the drip from dirt, etc., and provides an open space, which enables the hydrant to *drain* more freely.

199. The Walker hydrant * (Fig. 150) has a rubber-faced gate, which is moved up or down by a valve-rod which rests on the step-nut at the bottom of the hydrant chest. The valve-screw operates in a solid bronze wedge-nut which carries the valve up or down. The valve is constructed in such a manner that when it is opened it is first moved horizontally from its seat and is then lowered vertically out of the waterway. In closing the valve these motions are reversed. By this arrangement the rubbing and scraping of the valve surface is avoided. The motions of the valve are effected by the following patented device: when the valve is closed, a cam-lever rests on the cast iron webs or ways, shown in the cut with valve closed, and as the valve starts downward, this cam-lever drops forward into the circular groove in the valve and locks the valve and wedge-nut together. The valve then travels down vertically until it strikes the end of the valve-screw. The drip-plug, being attached to the wedge, enters the drip-sleeve and shuts off the drip. When the hydrant is being closed, the cam-lever pushes vertically on the main valve, and is held in this circular groove by the web in the back of the hydrant.

The main valve cannot push forward, until the cam-lever has reached the top of the webs. At this time the top of the valve comes in contact with the shoulder on the valve-screw and cannot travel upwards any farther; therefore, the cam-lever slides outward over the top of the webs, and the wedge-nut working upward, forces the valve directly forward.

Owing to the oblong hole in the valve, due to the horizontal movement of the same, the valve would tip forward, unless guided by two vertical ribs, cast on the webs mentioned above, which keep the valve in a vertical position at all times. The step-nut on the end of the valve-

* Manufactured by ing Co., Florence, Mass.

screw rests on the bronze step in the bottom of the hydrant. It takes the end thrust of the valve-screw, and protects the end of this screw. It can be taken off, when the valve is removed from the screw.

The draining of the hydrant, when closed, is effected by a drip-valve, which consists of a brass tube extending through the bottom of the hydrant. The drip-valve is attached to the back of the wedge-nut, so that the instant that the wedge-nut starts to travel down in the opening of

VALVE OPENED

LIST OF PARTS

Cast Iron

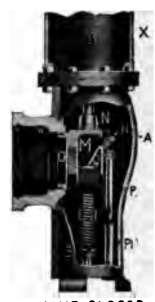
A — Chest.
B — Post.
C — Head.
D — Hose Cap.
E — Steamer Cap.
F — Top Cap.
G — Cover.
H — Gland.
J — Rod-head.
K — Wrench-head.
M — Valve.
W — Collar.
X — Frost-case.
XI — Frost-case Collar.

Wrought Iron

U — Cap Bolt.
V — Gland Bolt.
L—Valve-rod with Bronze Screw welded on end.

Bronze

LI — Valve-screw with End to weld on.
N — Valve Washer.
P — Drip-valve.
PI — Drip-sleeve.
Q — Wedge.
R — Cam-lever.
S — Step-nut.
T — Step.
O — Seat.



VALVE CLOSED

FIG. 150. — Walker Fire Hydrant.

the hydrant, the drip-plug enters the drip-sleeve, thereby closing off the drip.

The rubber-face of the valve is easily removed and replaced. With the exception of the upper part of the valve-rod, which is made of wrought iron, and the cast iron shell of the valve, all moving parts of the hydrant are made of solid bronze. All bolts have brass nuts. Wrench-heads and rod-heads are covered with brass tubing, so that all working parts are either of bronze or bronze bushed.

The head, post, and valve-case are made in separate parts, bolted together, faced off true, and drilled to a template. A collar protects the joint between the post and head. If the head or post should be broken, no leakage would occur, as the valve-rod would free itself from the wrench-head, and the valve would remain wedged on its seat. A frost-case and collar are furnished, when ordered.

A — Hydrant Barrel.
B — Hydrant Bottom.
C — Movable Seat-ring.
D — Upper Plate.
EE — Guide Wings.
F — Packing Strip.
G — Drain Outlet.
I — Double Drain.
J — Stem-nut.
K — Hold-down Nut.
L — Stuffing-box.
M — Stem.

FIG. 151. — The Columbian Fire Hydrant.

200. The Columbian Fire Hydrant* (Fig. 151). — This hydrant consists of a large barrel which is connected by a flange joint to the shoe (bottom piece). This makes it possible to place the nozzles in any desired direction. The main valve is of the compression type. It is provided with an upper plate *D*, of bronze, on which two guide wings *EE* are cast solid. The bronze seat-ring, which is screwed into the barrel

* Manufactured by the Columbian Iron Works, Chattanooga, Tenn.

at its lower end, has a drain outlet *G*, which is opened and closed, as the valve is moved up and down, by means of the leather packing strip *F*, which is dovetailed into one of the guide wings *E*. The annular passage *H* distributes the waste water. So long as the main valve is open, the outlet *G* remains covered, but so soon as the valve is closed, the outlet opens automatically and drains the barrel.

The main valve is opened and closed by means of its stem by turning the stem-nut *J*. The stuffing-box at the top of the hydrant can be repacked by merely unscrewing the hold-down nut *K*.

The working parts are all of bronze, and can be readily removed from the top of the hydrant, without disconnecting the hydrant barrel. The seat-ring can be unscrewed by means of a special wrench, and taken out through the top of the hydrant.

CHAPTER XIII

INTAKE-PIPES AND TUNNELS

201. General Conditions. — Cities situated on the shores of a lake, or on the banks of a river, often obtain their water supply by laying a line of pipes into the lake or river to a sufficient distance from the shore to insure the purity of the supply. By going far enough into a lake, a satisfactory water supply may be obtained in this manner; but, in the case of rivers, the water has usually to be purified by filtration. For large cities, tunnels are driven under the bottom of lakes or rivers, to obtain the desired quantity of water supply.

The inlet of the intake must be properly protected by a crib, gate-house or tower, provided with fish-screens, and with suitable gates, placed at different elevations, so as to make it possible to draw water at different levels from the lake or river, as may be desired. In cold latitudes the inlet ports are apt to be obstructed by anchor ice (p. 230). This trouble may be largely obviated by making the inlet ports about 4 to 8 times the area of the inlet conduit.

In order to avoid the loss by friction, which would occur when a long intake-pipe has to act practically as a suction-pipe, a wet-well or receiving-chamber is often constructed near the pumping station. If the supply is drawn from a river subject to great fluctuations in the level of its water surface, the intake must be placed at an elevation that will enable it to obtain a supply at *low-water*. This may necessitate the placing of the pumps in a deep, water-tight well, to enable them to suck the water.

202. Intake-pipe of Erie, Penn.* — In 1868 the city of Erie, Penn., built an intake pipe-line, 975 ft. long, into Erie Harbor, a bay of Lake Erie, having an area of about $5\frac{1}{2}$ square miles. This line consisted of a combination of brick, wood, and wrought iron conduits. The harbor soon became so polluted by sewage that the intake pipe-line had to be extended further into the harbor. This was done in 1896 by laying a line of 5-ft. cast iron pipes, 8307 ft. long, on the bottom of the harbor to a new intake-crib, located within 1500 ft. of the outer limits of deep water in the harbor. In about ten years the new intake became unsatisfactory, on account of sewage contamination, and the pipe-line was extended into Lake Erie by a 5-ft. riveted steel pipe. The total length of the intake

* *Eng. News*, Dec. 5, 1895, p. 373.

conduit is now 17,637 ft., and the inlet is formed by a crib, located in the lake, 5430 ft. from a peninsula that separates the Erie Harbor from the lake. The water at this crib is practically free from pollution. At certain seasons of the year, however, there is considerable turbidity, due to the action of the waves in shallow water near the shore, and it is proposed to build settling basins to remove the greater part of the sediment.

The pipe laid in the last extension of the intake-line was made of ½-in. soft-steel plates, which were usually riveted together at the shop in lengths of 30 ft., and then coated with mineral rubber asphalt pipe coating, applied at a temperature of 300 degrees F. The pipe was constructed with taper rings or courses, the small end of each ring being inserted in

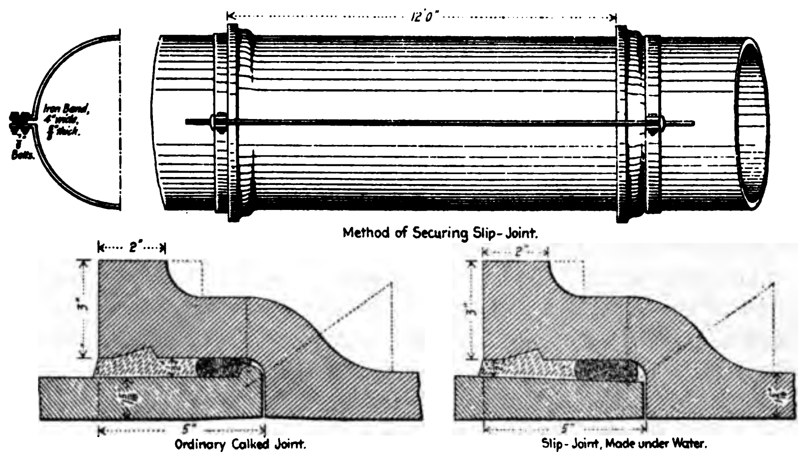


Fig. 152. — Intake-pipe of Erie, Penn.

the large end of the next ring. Each course has but one longitudinal joint, which is a double-riveted lap-joint, while the circular joints are single-riveted lap-joints. The pipe was shipped in 30-ft. lengths to Erie, and several of these lengths were riveted together in the field, so as to form sections of 120 or 150 ft. All joints were carefully painted with Smith's durable metal coating.

At the ends of each section of pipe, a special submarine joint (Fig. 152) was attached, except at connection with *specials*, where faced, welded steel, angle-flanges were riveted to the pipe. The submarine joint is a slip-sleeve joint, formed by a circular plate and an angle on the end of one section, and a loose angle and a circular lug riveted on the adjoining end of the next section. One angle is riveted in the shop on the end of a section, and the circular plate is riveted on the outside of it, forming a sleeve extending beyond the end of the section. The attached angle is fitted with 16 bolts, extending longitudinally with the pipe, about $6\frac{1}{2}$ ins. beyond the outer end of the circular sleeve-plate. These bolts are threaded at the outer end, and carry the second circular angle, which is free from both sections and from the bolts. The end of the other section, to which the lug is riveted, slips into the end of the section carrying the sleeve-plate, the edges of the lug on the one section and of one leg of the attached angle on the other section abutting. The horizontal leg of the loose angle follows into the sleeve, after the lug and the space between its end and the lug is filled with a 1-in. lead pipe, which is flattened down, before the joint is made. As the sections are drawn together by tightening the bolts, the lead pipe takes the shape of the opening in the joint, and fills it. Special joints were used where the pipe-line had to be connected to special pipes.

Several expansion joints were placed at different points in the pipe-line. These joints were made by inserting, between two regular land joints, a short length of pipe with an angle attached to one end, the other end being smooth. The end carrying the angle was joined to the end of a regular length by a standard land joint, but the smooth end was slipped into the end of another regular length of pipe, until it lapped $2\frac{1}{2}$ to 3 ft., the space between the two pipes being $\frac{3}{16}$ in. wide. The leg of a loose circular angle around the short expansion joint length of pipe was then bolted to the vertical leg of the angle riveted to the end of the regular length of pipe, the horizontal legs of the two angles extending in opposite directions in the same plane. Bevels were cut on the abutting corners between the angles, so as to form with the outside of the inner pipe an annular space, irregularly triangular in cross-section, in which a $\frac{5}{8}$-in. lead pipe was placed. This pipe was then squeezed into the annular space, by drawing the loose angles, by means of the bolts, as close as possible to the attached angle, thus making a tight joint.

The pipe was laid in a trench, and covered with the material excavated. The depth of the trench varied from 8 to 21 ft. below the surface of the ground. The depth of water at the intake-crib is about 44 ft. In dredging the trench for the steel pipe extension, very soft material was encountered at the old crib, which made the progress very slow. A 20-in. centrifugal dredge was finally tried, and proved to be satisfactory. The submerged part of the steel pipe was laid in the dredged trench in the following manner:

The ends of each section of pipes (120 or 150 ft. long) were closed by bulkheads, fitted with air-locks, and the sections were then launched into the water from ways. The sections floated, with more than half of the pipe out of water. They were easily towed into the proper position over the trench, and held in place by temporary piles, which had been previously driven. Two piles were placed on each side of both ends of each section. The piles were capped with heavy timbers, transversely of the pipe-line, and were thoroughly cross-braced. Two timber foundations were secured to the pipe, 19 ft. either way from the joint. They were held in place by hooks connecting the foundation platform to a saddle piece, fitting over the top of the pipe. The bulkheads were then removed, and the pipe was lowered to the bottom of the trench, by means of ropes from swinging booms. A diver guided the spigot into the bell of the last pipe laid, adjusted the hook-bolts, and by screwing up the nuts on the same, upset the lead pipe, thus completing the joint. The saddle pieces on the pipe were then removed, and used for the next pipe.

The intake of the pipe-line is formed by a timber crib, 40 ft. square and 19 ft. high. It has a flat top and bottom, and vertical sides. Its top is submerged 25 ft. The four sides of the crib are built of 12 × 12-in. hemlock timbers. Two longitudinal and two transverse rows of 12 × 12-in. timber divide the interior of the crib into 9 pockets, each 12 ft. square, which form three rows. A layer of 4-in. planks is laid over, and spiked to, the bottom layer of timbers, and a layer of 3-in. planks is placed over the 4-in. layer and spiked to it. The first layer of plank is laid diagonally with the crib, and the second layer is laid in the opposite diagonal direction.

The intake-pipe extends through the crib on the bottom of the middle transverse row of pockets, and is blanked at its outer end. A vertical connection, 6 ft. long, extends up from the pipe in the center pocket, and is open at the top. The crib is held in place by rock dumped into the four corner pockets. The two inner sides of these pockets are lined with vertical 3 × 12-in. planks, having a 2-in. notch, cut in both sides of each plank, for a depth of 4 ft. from the top. The other five pockets are covered with wooden gratings. The crib was built on a dock on the water front, launched into the water, towed to the proper position, and sunk by filling the corner pockets with rock.

The flow through the pipe-line is controlled by three 60-in. gate-valves, placed in the pipe-line, where it crosses the peninsula that separates Erie Harbor from the lake. One of these valves is on the lake side, one is on the land side, and the third is between the other two. The soil of the peninsula consists principally of fine sand. At the sites of the valves, the water level is at, or very close to, the ground line. This would have made the construction of masonry valve-chambers very difficult, and a timber valve-well was, therefore, built for each gate-valve. Each of these wells is about 8 by 11 ft. in plan and 17 ft. deep. The ends, sides, and bottom of the well are built entirely of 8 × 8-in. timbers, an

8 × 8-in. post being placed in each corner. The joints between the timbers overlap alternately at the corners. After the joints had been thoroughly caulked, the whole well was sheathed with $\frac{7}{8}$-in. tongue-and-grooved maple flooring.

Two branch connections, each 130 ft. long, were laid on the peninsula, 680 ft. apart, on one side of the pipe-line. Each of these connections, which are to lead to settling basins that are to be built in the future, is closed by a 60-in. gate-valve.

203. Intake-pipe at Milwaukee, Wis.* — In 1874 Milwaukee obtained a water supply from Lake Michigan, by laying a line of 36-in. cast iron pipe in a trench on the bottom of the lake to a crib, located 2100 ft. from the shore. The supply thus secured soon became insufficient, on account of the growth of the city, and in 1889 it was decided

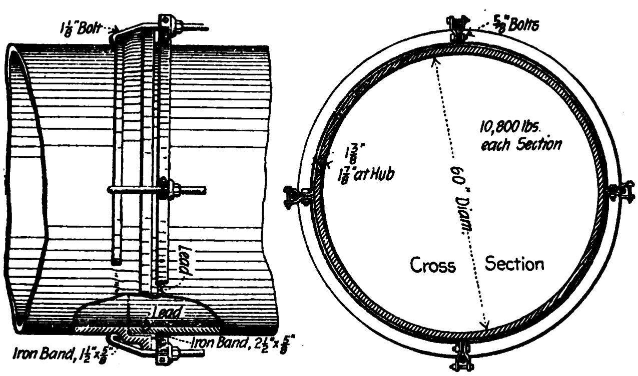


FIG. 153. — Intake-pipe of Milwaukee, Wis.

to construct an intake of larger capacity. This was to be formed by a brick-lined tunnel, 7½ ft. in diameter, for a distance of 3200 ft. from a shaft sunk near the shore, and the intake was to be extended for an additional distance of 5000 ft., by means of two lines of 60-in. cast iron pipes, to an intake-crib sunk in the lake, which was to protect the entrance to the pipe-lines. The total length of the intake conduit was, therefore, to be 8200 ft.

The driving of the intake-tunnel proved to be one of the most difficult works of its kind that has thus far been undertaken.† The two lines of 60-in. cast iron pipe were laid, with comparative ease, in a dredged trench, from the lake-shaft at the end of the tunnel to the sunken crib at the end of the intake-line. Four pipes, forming a section 50 ft. long, were connected on shore, and after the ends had been closed by tem-

* *Eng. News*, Sept. 19, 1895, p. 187.
† *Ibid.*, pp. 187–190.

porary caps, each section was towed by a barge to the point where it was to be sunk. The spigot-end of each 50-ft. section was fitted with a temporary bell, and poured with lead before it left the shore. The temporary bell was then removed. After the section of pipe had been sunk, and the spigot-end, with its lead joint, had been entered into the bell of the preceding section laid, divers pulled the joint tight, by means of the clamps shown in Fig. 153. In this manner as much as 200 ft. of pipe was laid per day. An octagonal crib, Fig. 154, built of 12 × 12-in. timbers, forms the inlet to the two pipe-lines.

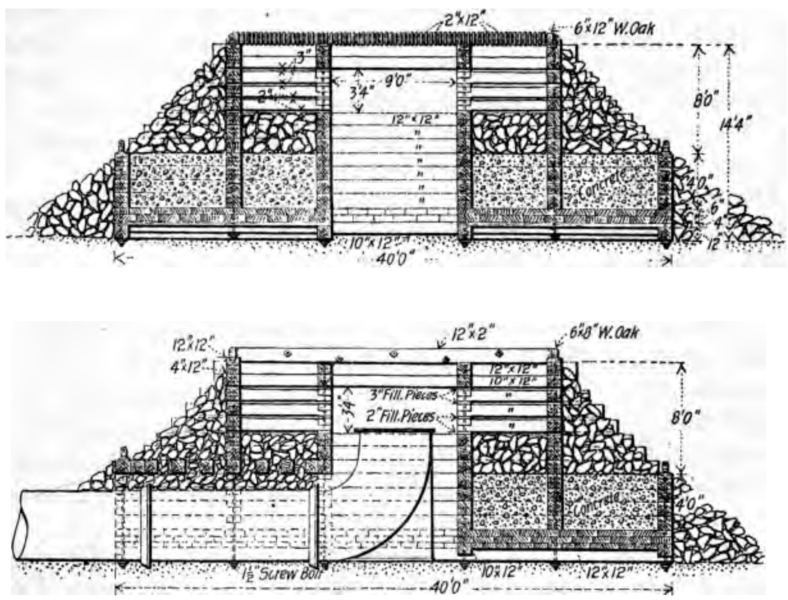


FIG. 154. — Intake-crib of Milwaukee, Wis.

204. Intake-pipe of Toronto, Ont.* — The city of Toronto obtains its water supply from Lake Ontario through a long intake-pipe which crosses the harbor. In 1889 the water was drawn from the lake at a distance of about 2400 ft. from Hiawatha Island, where the depth of the water is about 30 ft. The intake consisted of a wooden crib, 40 ft. square, divided into nine compartments, all of which, with the exception of the center one, were filled with stone. The water entered at the top of the crib, passed through a 6-ft. wooden pipe to a crib, 2357 ft. distant on the Island, from which it flowed through a 4-ft. wooden pipe to another crib at Hanlan's Point, a distance of 6010 ft., and was then con-

* *Eng. News*, Oct. 17, 1891, p. 366, and June 29, 1893, p. 614. Also, annual reports of the Superintendent of the Water Works of Toronto, for 1889 to 1893.

veyed by a 3-ft. cast iron pipe, 4600 ft. long, to the pump-well on shore. The total length from the intake-crib to the pump-well was about $2\frac{1}{2}$ miles.

In 1889 it was decided to lay a 48-in. steel pipe along the 3-ft. cast iron pipe from the pump-well to Hanlan's Point, and to replace the 4-ft. wooden pipe from this place to the intake by a 5-ft. steel pipe. It was, also, decided to extend the intake-pipe 350 ft. farther into the lake to a depth of 60 ft., by laying a 6-ft. steel pipe. This work was done in 1890 to 1891.

The 4-ft. steel pipe was laid for a distance of 4660 ft., almost parallel with the old 3-ft. cast iron pipe from the pump-well on Hanlan's Point, where there is a depth of 14 to 24 ft. of water. A water-tight chamber, $8\frac{1}{2}$ ft. square, was constructed in the center of the connecting crib. It is fed by a 5-ft. steel pipe, 6027 ft. long, which was laid to replace the old 4-ft. wooden pipe from the connecting crib to the intake-crib.

The steel pipes laid in this intake-line are all provided with the flexible joints designed by Wm. H. Law, C. E. (p. 149).

205. Intake-pipe of Syracuse, N. Y.* — The water supply of Syracuse, N. Y., is obtained from Skaneateles Lake by means of a 54-in. steel intake-pipe which was laid on the bottom of the lake for a distance of about 4500 ft. from the shore to an intake-crib. The pipe was made of $\frac{3}{8}$-in. steel plates, weighing 15 lbs. per square foot. Each plate formed a 6-ft. length of pipe, slightly larger at one end than at the other, so that it could be telescope-jointed with the adjoining pipes. Five pieces of pipe, riveted together with $2\frac{1}{2}$-in. lap, formed a section, 29 ft. long, which was coated with asphalt.

On the lake shore, four 29-ft. lengths of pipe were riveted together, making a section, about 116 ft. long, which was provided with a steel spigot at one end, and a cast iron hub at the other. The hub was provided with 20 steel hook-bolts, $1\frac{1}{2}$ ins. in diameter, with hexagonal nuts. A gasket of 1-in. soft lead pipe was placed around the pipe against the steel spigot, and a $\frac{7}{8}$ by 1-in. wrought iron hoop was placed back of the lead gasket, as a follower and bearing for the hook bolts.

The flexible joints (Fig. 155) were made by jointing two short pieces of pipe together. One of these pieces was 3 ft. long and tapered in diameter. The other was 4 ft. long and straight. Upon the end of the latter, a machine-faced cast iron ball or zone was riveted. Two 4-in. channel irons were riveted on the inside of the larger end of the tapering piece, which fitted over the ball. The channels were run full of hot lead against the ball, thus forming a flexible joint, which could be deflected 12 degrees from the axis of the pipe in any direction.

* Trans. Am. Soc. C. E., 1895, XXXIV, p. 28.

The pipe was laid in a trench, dredged in the lake. The ends of each of the 116-ft. sections of pipe were closed by oiled canvas bulkheads.

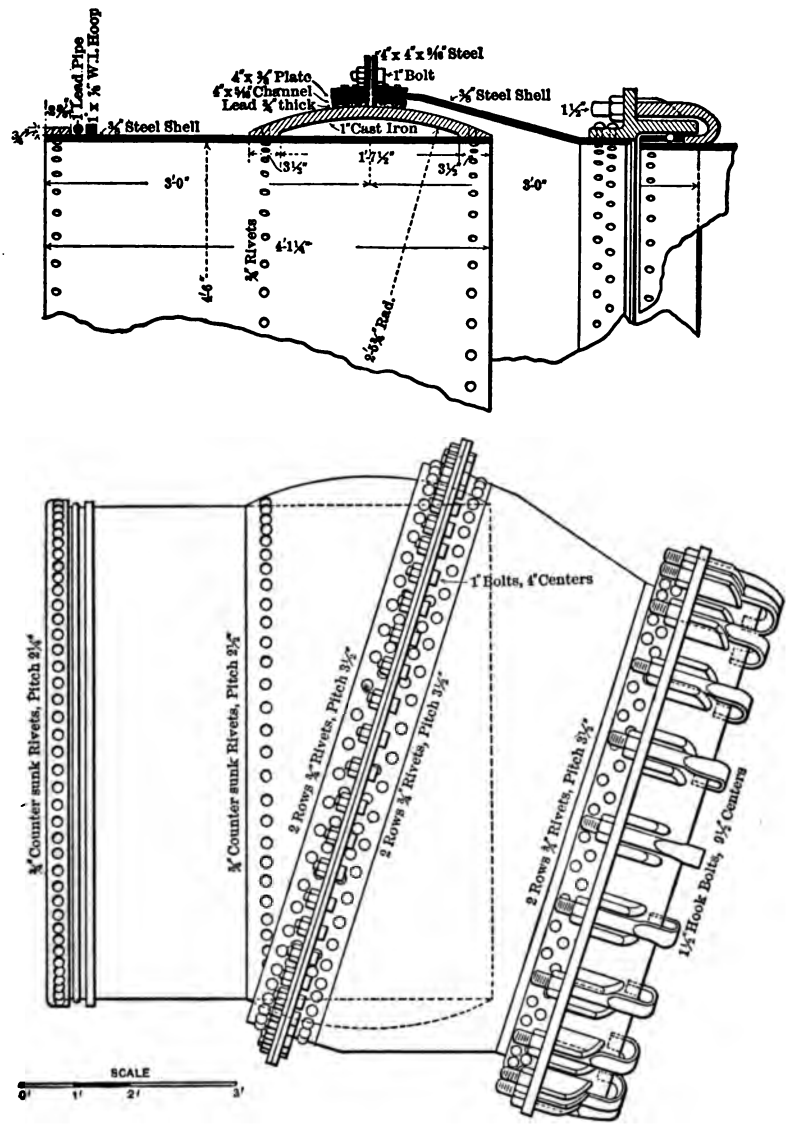


FIG. 155. — Intake-pipe of Syracuse, N. Y.

Each section was rolled into the water, and floated between the sections of a catamaran, which had been placed over the pipe trench, and was held in position by spud-piles at each corner. Two timber foundation platforms were secured to the pipe, 19 ft. either way from the joint. They were held in place by hooks connecting the foundation platform to a saddle piece, fitting over the top of the pipe. The bulkheads were then removed, and the pipe was lowered to the bottom of the trench, by means of ropes from swinging booms. A diver guided the spigot into the bell of the last pipe laid, adjusted the hook-bolts, and by screwing up the nuts on the same, upset the lead pipe, thus completing the joint. The saddle pieces on the pipe were then removed, and used for the next pipe.

206. Intake-pipe of Rochester, N. Y.* — A riveted steel pipe, 5 ft. in diameter and about 1500 ft. long, was laid in 1894 in Hemlock Lake, N. Y., to obtain a supply for the second gravity conduit of Rochester, N. Y. The pipes were riveted together in lengths of about 100 ft., and provided at the ends with the component parts of the ball-and-socket joints shown in Fig. 156. These flexible joints admit of a deflection of $17\frac{1}{2}$ degrees in any direction. They are made of steel plates, stiffened with bar, angle, and channel rings, cast iron being only used for the spherical zones or ball portions. The bearings for the spherical surface are formed of lead, which was melted and poured into the channel rings of the socket and collar, after the parts had been assembled in the shop.

In the Rochester intake the Law joint (Fig. 89, p. 150) was modified by increasing considerably the length of the chord subtended by the two lead bearings, by using angle rings of unequal sides. The joint was also greatly strengthened by the introduction of the heavy steel reinforcement rings, above or around the bearings. Each of these flexible joints weighed about 6200 lbs., complete, the weight of the cast iron zone or ball portion being about 2700 lbs. The joints were made in Peterborough, Ontario, and each cost, attached to the pipe at Hemlock Lake, not including the United States Customs duty, $290.

The 5-ft. steel pipe was made of $\frac{3}{8}$-in. plate, stiffened with $3 \times 3\frac{1}{2} \times \frac{1}{2}$-in. steel angles where it was to be covered with heavy back filling, and of $\frac{5}{16}$-in. plate where the back filling was light. The pipes were laid in water, 20 to 35 ft. deep.

207. Intake-pipe of Duluth, Minn.† — In 1897 and 1898 the city of Duluth obtained a new water supply from Lake Superior, by laying a steel pipe, 60 ins. in diameter and 1500 ft. long, in the lake at a point on the north shore, situated about 8 miles east of the City Hall. Excellent water, varying in temperature only a few degrees during the year, was thus obtained. In midsummer the temperature of the water is about 40° F. The pipe was made of $\frac{3}{8}$-in. steel in sections, 116 ft. long,

* Trans. Am. Soc. C. E., 1895, XXXIII, p. 271.

† *Eng. News*, May 25, 1898, p. 282.

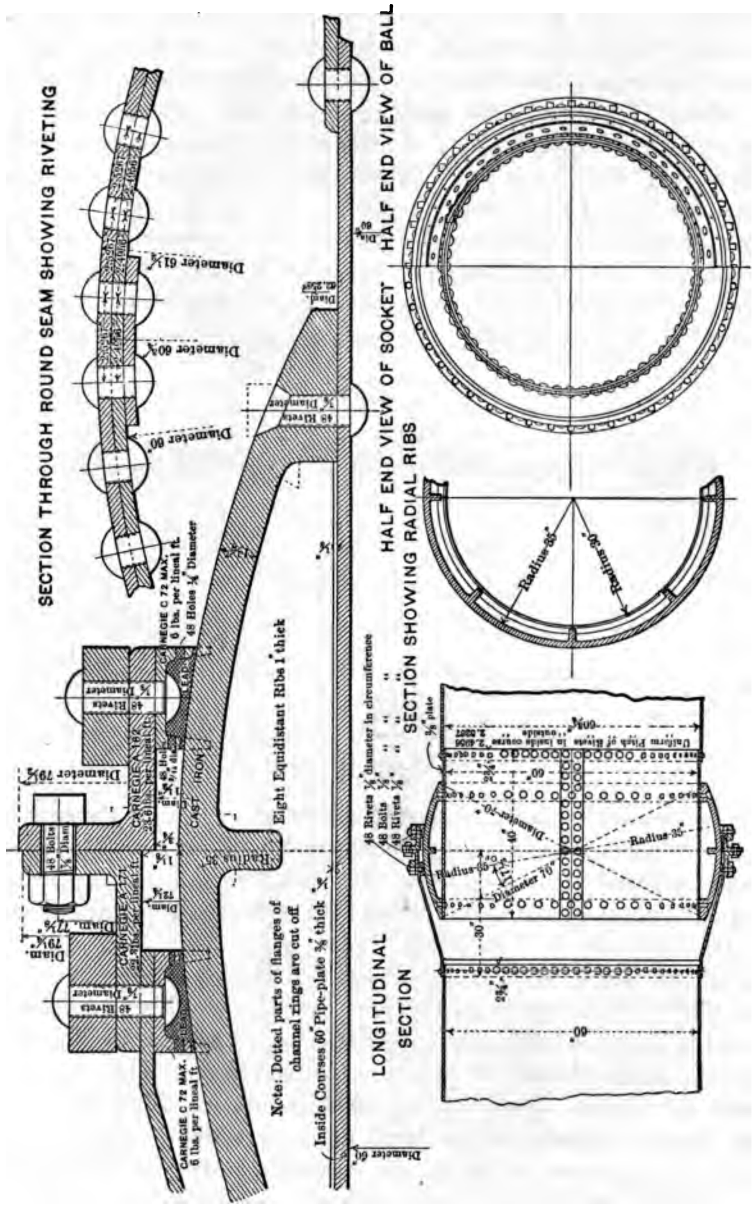


FIG. 156. — Intake-pipe of Rochester, N. Y.

which were joined under water by means of rigid and flexible joints, made tight by a caulking of lead. Four flexible joints (Fig. 157) were placed in the pipe-line: one at the junction of the pipe with the intake-well, one at the submerged intake-crib, and the remaining two at changes of grade. At its end, the pipe is turned upwards, by means of an elbow, so as to take water about 11 ft. above the bottom of the lake. The elbow is protected, and the water entering it is screened, by means of a submerged inlet-crib (Fig. 158), sunk in 67 ft. of water. The area of the spaces between the bars of the screen is ten times that of the pipe.

The pipe enters the intake-well on shore, 20 ft. below the level of the lake, and the flow is regulated here by a 60-in. Eddy double-parallel-seat valve. For about 814 ft. from the shore, the pipe is laid in a trench excavated in the rock bottom. Beyond the pipe trench, the pipe rests

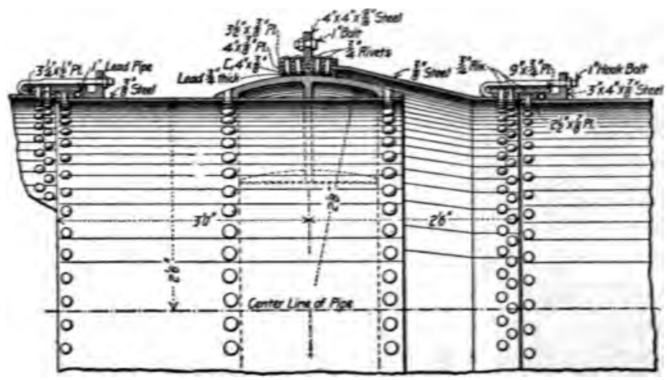


FIG. 157. — Intake-pipe of Duluth, Minn.

on the bottom of the lake, which was properly leveled up to receive it. Five anchor cribs were placed at equal distances between the intake-crib and the point where the pipe left the rock trench. These cribs were loaded with broken rock, and covered with rocks, each containing 60 to 80 cu. ft.

The intake-well is 20 ft. in diameter and 20 ft. deep, below the surface of the lake. It was excavated in rock and was lined with brickwork. The well is provided with a double set of copper screens with $\frac{3}{4}$-in. mesh. The pipe cost $9.11 per foot, delivered on the skids. The rigid joints cost $82 per piece and each of the flexible joints cost $398.25.

208. Intake-tunnels. — For large cities, a tunnel must be used, in place of a pipe-line, to obtain the desired quantity of water from a lake or river. The inlet to such a tunnel is controlled by means of a crib, or gate-house, similar in construction, but larger than what is required for a pipe-line.

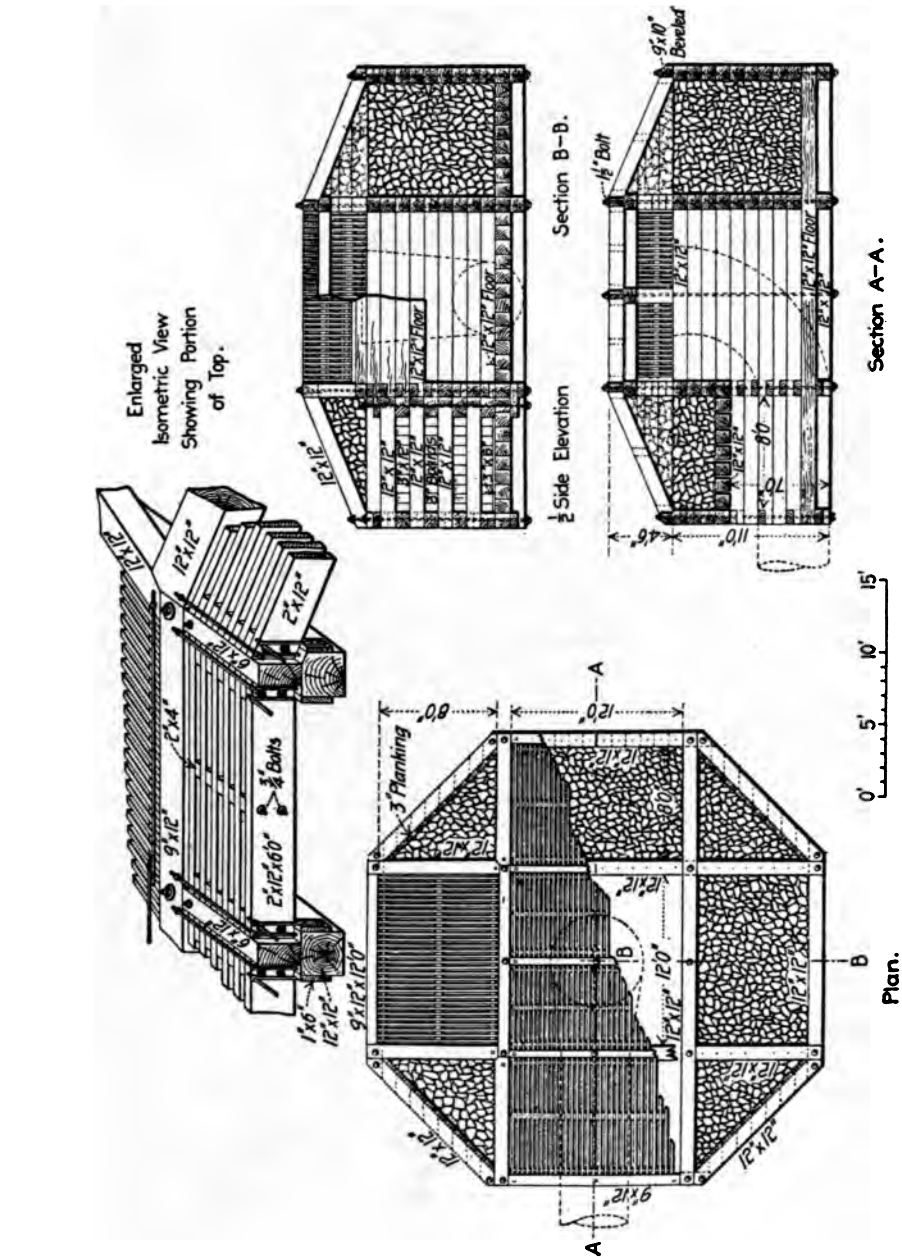


FIG. 158. — Intake-crib of Duluth, Minn.

209. *Chicago, Ill.*, obtains a supply of about 1,000,000,000 gallons per day from Lake Michigan, by means of nine tunnels, constructed under the bottom of the lake in a stratum of compact blue clay, or in limestone. The first supply for the city was obtained in 1836 by laying an iron pipe on the bottom of the lake, for a distance of about 150 ft. The size of the tunnels which now supply the city is given in the following table:

TABLE XL. — INTAKE-TUNNELS OF CHICAGO

Completed	Cross-section	Length in miles	Lining
1867	5 ft. in diameter	2	8 ins. brickwork
1875	7 " " "	2	8 " "
1892	6 and 8 " " "	6.5	13 " "
1895	7 " " "	2	13 " "
1896	6 " " "	2	13 " "
1898	5 and 7 " " "	3.9	13 " "
1898	10 " " "	2.7	13 " "
1911	14 " " "	2.3	Concrete
1918	horse-shoe section	3	"

The nine intake-tunnels supply four large pumping stations near the lake, and feed seven land tunnels which supply six inland pumping stations. The land tunnels are 5–12 ft. in diameter, and have an aggregate length of 33 miles. They were constructed for cross-connections between the pumping stations and intake-tunnels, in order to insure a constant supply.

The intake-cribs at which the tunnels draw their supplies are located 2 to 4 miles from shore, in water from 26 to 37 ft. deep. At that depth, the sand that overlies the clay stratum under the lake is not moved much by the water, but considerable trouble has been experienced on account of anchor ice.*

There are seven intake-cribs, the first three tunnels constructed drawing their supplies from the same crib, which was built in 1865, two miles from shore. It is a pentagonal timber structure with sides 58 ft. long and a height of 40 ft. The crib is surrounded by a protecting stone breakwater, having a 30-ft. opening to admit boats. A wooden super-

* Anchor ice consists of thin scales of ice which form in moving water and are readily carried below the surface by weak currents of water. It adheres to submerged surfaces, and soon forms large blocks of ice which it is difficult to remove. Steam, water or air under pressure, or mechanical means are used for this purpose. Anchor ice forms often in northern rivers where the current is rapid, but not in quiet water that is covered by ice. In order to avoid clogging by anchor ice, the ports of an intake should be four to eight times the area of the inlet conduit.

structure was originally built on top of the crib, but was replaced in 1874 by a structure of stone masonry with a lighthouse and a house for the keeper. The crib has three inlet-ports, 4 ft. wide by 5 ft. high. There are two intake-shafts in the crib, and one between the crib and the breakwater. The shaft which supplies the 5-ft. tunnel is 8 ft. in diameter. It is lined with brickwork having a 9-ft. cast iron cylindrical top, and has two 4- by 5-ft. sluice-gates.

The second shaft, which supplies the 7-ft. tunnel, is of similar construction, but has three 4- by 5-ft. sluice-gates. The third shaft, which is located between the crib and the breakwater, supplies the third lake tunnel. It is 10 ft. in diameter, and is lined with brickwork surmounted by a cast iron top. This shaft has three 4- by 5-ft. sluice-gates, and is filled with stone.

The second intake-crib was built in 1892, four miles from shore. It is a steel caisson, having two circular steel cylinders, $\frac{3}{8}$ in. thick, with diameters, respectively, of 70 and 120 ft. There are twenty-four radial bulkheads between the two shells, and the annular space is filled with concrete. The steel caisson rests on a polygonal timber grillage, 13 ft. high, having a solid timber bottom, 2 ft. thick. Six gated ports, 5 by 5 ft. square, admit the water to the central well. The intake-shaft is 10 ft. in diameter, and has two 5- by 6-ft. sluice-gates. A masonry superstructure is built on top of the caisson.

The seventh crib,* built in 1915 to 1918, differs in design from those previously constructed. The older cribs were all provided with wooden bottoms, in order to make it possible to float them into place and to sink them. These cribs were top-heavy, and rocked perceptibly during severe storms. With this type of crib the intake-shaft could not be sunk until the solid timber bottom had been cut through.

In the plans for the seventh crib, the wooden bottom was omitted. The crib (Fig. 159) consists of two concentric steel shells, the outer shell being $\frac{3}{8}$ in. thick and 90 ft. in diameter, and the inner one 40 ft. in diameter. The two shells are held together by intermediate bracings and the intake-ports. By bulkheading the ports and using intermediate flotation chambers attached to the bracing, the steel shells were easily floated. They were provided on the bottom with a cutting edge, to enable the crib to seal itself into the bed of the lake, thus permitting the pumping out of the intermediate well, when the crib was sunk into position.

In this crib the gates and the part of the cast iron cylinders extending above the surface of the lake, used with the earlier cribs, were dispensed with, and the top of the intake-shaft was placed about 10 ft. below the

* Jour. Western Soc. of Engineers, May, 1916.

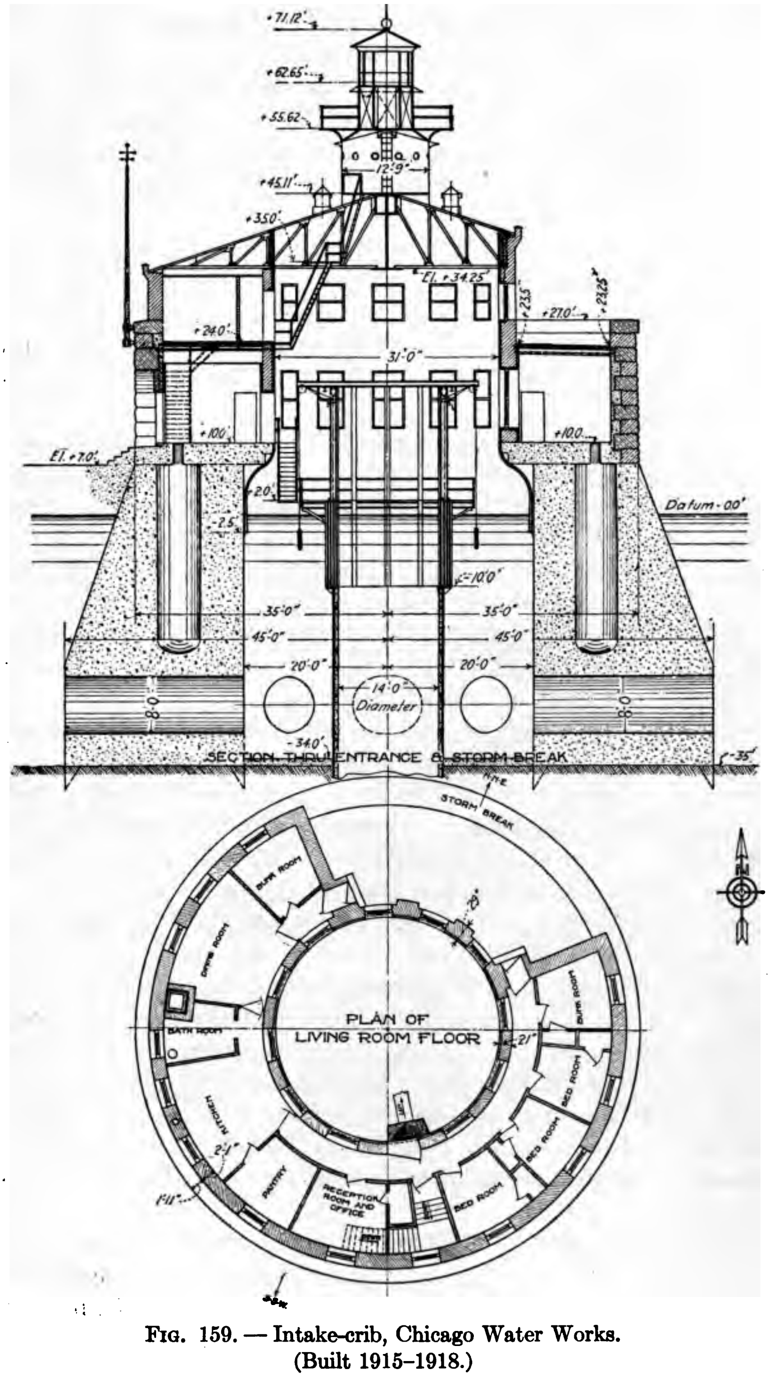


FIG. 159. — Intake-crib, Chicago Water Works.
(Built 1915–1918.)

surface of the water, leaving the entire area of the upper part of the shaft free for fish-screens.

210. *St. Louis, Mo.*, takes its water supply from the Mississippi River at the Chain of Rocks, about 10 miles up-stream from the business center of the city. The water is obtained from the river by means of two intake-towers and tunnels, and is pumped into settling basins.

The first intake-tower, built in 1889, was founded on rock, near the west side of the main channel, 1555 ft. from the west shore. It is a granite pier, 50 ft. high, which is provided with wells into which water can be drawn from the river through sluice-gates, placed at different levels. At the side of this intake, the depth of the river varies from 12 to 44 ft., and the current ranges from 4 to 6 ft. per hour.

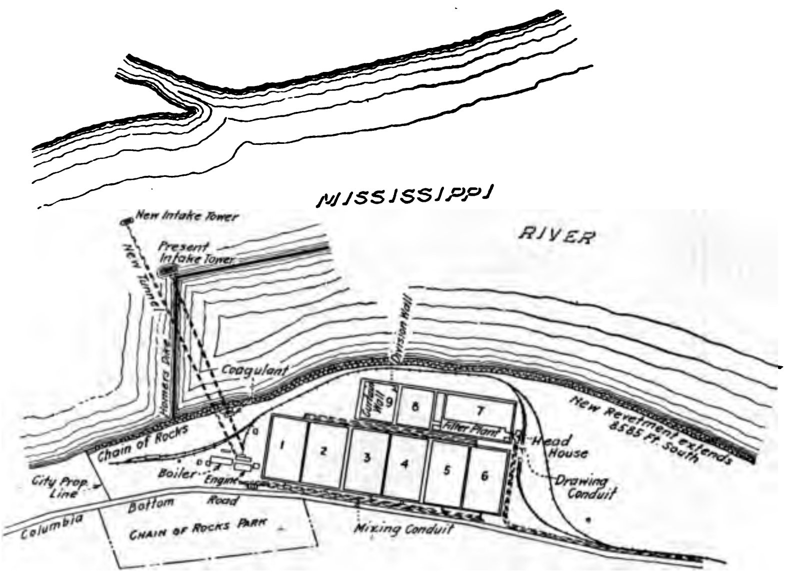


FIG. 160. — Intake-tunnels of St. Louis Water Works.

A brick-lined tunnel, 7 ft. in diameter and about 1500 ft. long, is connected with a 40-ft. shaft at the inlet-tower and with an uptake-shaft, just above high-water mark, on shore, to which the pump shaft is also joined. The water works, of which the intake and tunnel form part, were constructed in 1889 to 1893.

The second intake-tower and tunnel * were constructed in 1913–15, in order to provide additional means of obtaining water, in case the first tower should have to be shut off for repairs, etc. The second intake-tower is 700 ft. east, and 200 ft. north, of the first tower (Fig. 160).

* Annual Report of Water Commissioner, City of St. Louis, for the year ending April 1, 1915, p. 34.

It is designed as a bridge pier, to resist wind, ice and current. The tower (Figs. 160*a* and 160*b*), which is founded on rock, consists of a concrete substructure, faced on the nosing and below Elevation 80 with Georgia gray granite, and elsewhere with buff Bedford limestone. Be-

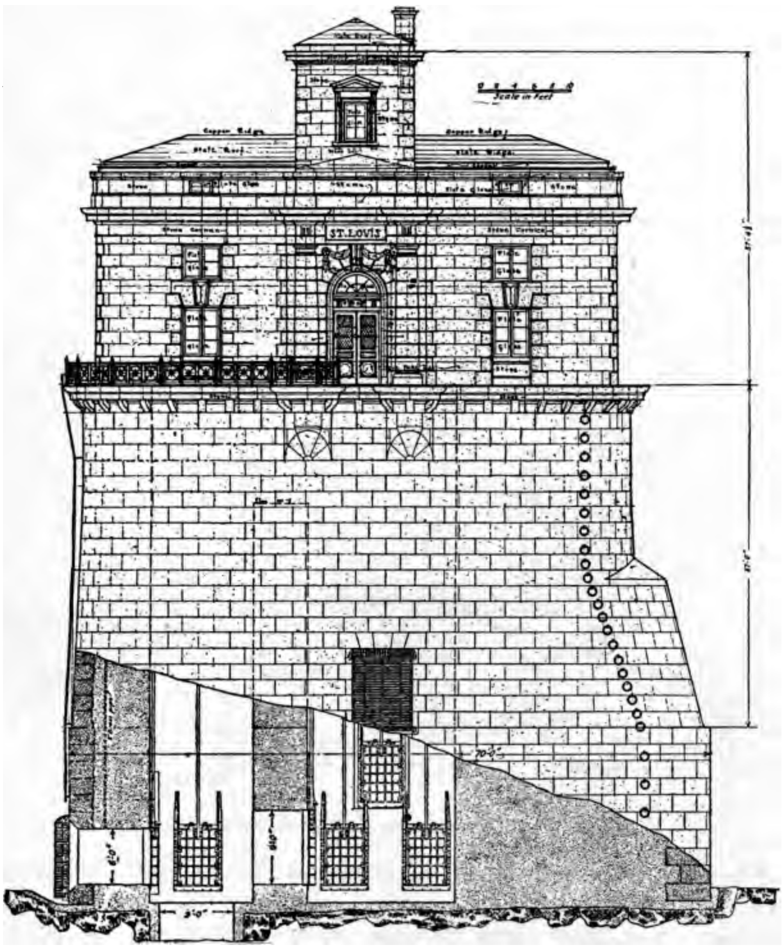


Fig. 160*a*. — Intake-tower of St. Louis, Mo.
(Built 1913–1915.)

low Elevation 80 the substructure is 70 ft. long by 26 ft. 8 in. wide. This part has vertical walls, a triangular nose, and semioctagonal back. For the next 26 ft. the back is semicircular, the sides batter $\frac{1}{2}$ in. per ft., and the nose slopes back to form an ice breaker, 18.3 ft. high, above which the front of the tower is semicircular. A stone balcony supported on stone brackets, and provided with an ornamental iron railing, overhangs the substructure.

The superstructure, which is designed on the style of Imperial Roman architecture, rises above the substructure to a height of 41.2 ft. It con-

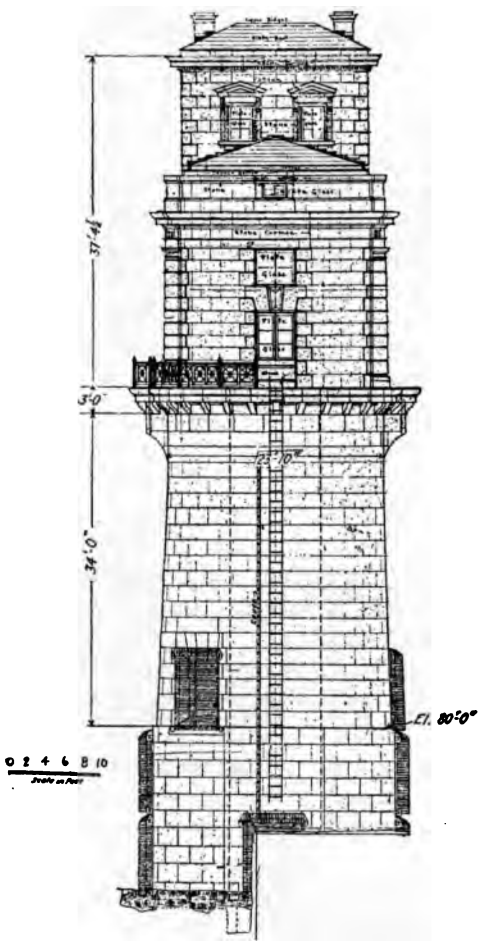


FIG. 160*b*. — Intake-tower, St. Louis Water Works.
(Built 1913–1915.)

tains the operating chamber, balconies with sleeping quarters for the operating forces, and a room above them for the tower keeper. All of its walls are of buff Bedford limestone, faced on the interior with white enameled bricks. The roof is of concrete, covered with heavy green slate.

In the interior are two wells, below the operating floor: one, 16 by 8 ft., rectangular in section, and the other, semioctagonal and semirectangular, 11 by 10 ft. Both wells extend to rock, and are connected at the bottom by a 6- by 8-ft. opening, controlled by a gate. The downstream well, which is the smaller of the two, centers on the down-take shaft. Ports are constructed in the substructure at different levels, to admit the river water. They are fitted with cast iron gratings, to prevent the entrance of large blocks of ice, logs, sticks, etc. As a further precaution, 2-in. brass pipes, imbedded in the concrete, extend from the operating floor to the upper surface of each of the nine lower ports. Steam, air, or water may be forced through these pipes, to remove any material clogging the openings. Each port is provided with a sluice-gate, which is located in the well and is operated by hydraulic cylinders at the operating floor.

A tunnel, 8 ft. in diameter, driven through limestone, connects the tower with the wet well. A shore-shaft, near the river bank, divides it into two sections, 2193 and 556 ft. long, respectively. The longer section, known as the river tunnel, drains to the shaft at a slope of 1 to 1000. The inshore tunnel rises on a 6 per cent grade to a screen-chamber, which is connected by a 90° bend with the wet-well of the pumping station.

211. *Cincinnati, O.,** obtains its water supply from the deep channel of the Ohio River, which is near the Kentucky shore. The intake, which is a substantial masonry pier, founded on rock, was located some distance above the city, to avoid pollution. At high-water, the depth of the river at this point is about 90 ft.

The intake-pier † (Fig. 161) is supported on a timber caisson, 57 by 29 ft. in plan, and 16 ft. high, which was carried down through sand and gravel to bed-rock. At the top of the caisson the pier is 55 by 27 ft. in size, and it tapers upward with a batter of 1 in 24 to elevation 83 above datum, where it is topped off with a course of coping-stone. On the up-stream end, the pier forms a pointed arch, and on the down-stream end it is semicircular.

The pier contains two wells: an inlet-well on the up-stream side, and a shaft-well on the down-stream side. The former is 9 by 14 ft. in section, and has its bottom at elevation −7.50. The latter, which forms the upper part of the tunnel-shaft, is half circular and half rectangular in section. The inlet-well has four rectangular ports, each 4 by 6 ft. in size. The bottom of the two ports in the channel are at elevation −5.50, while the bottom of the two ports on the shore side are at elevation +12.50.

* *Eng. Record*, Nov. 27, 1897, and July 13, 1901, p. 26.

† Report to the Trustees of the City of Cincinnati, Ohio, by Geo. H. Benzenberg, Chief Engineer of Water Works, 1909, p. 45.

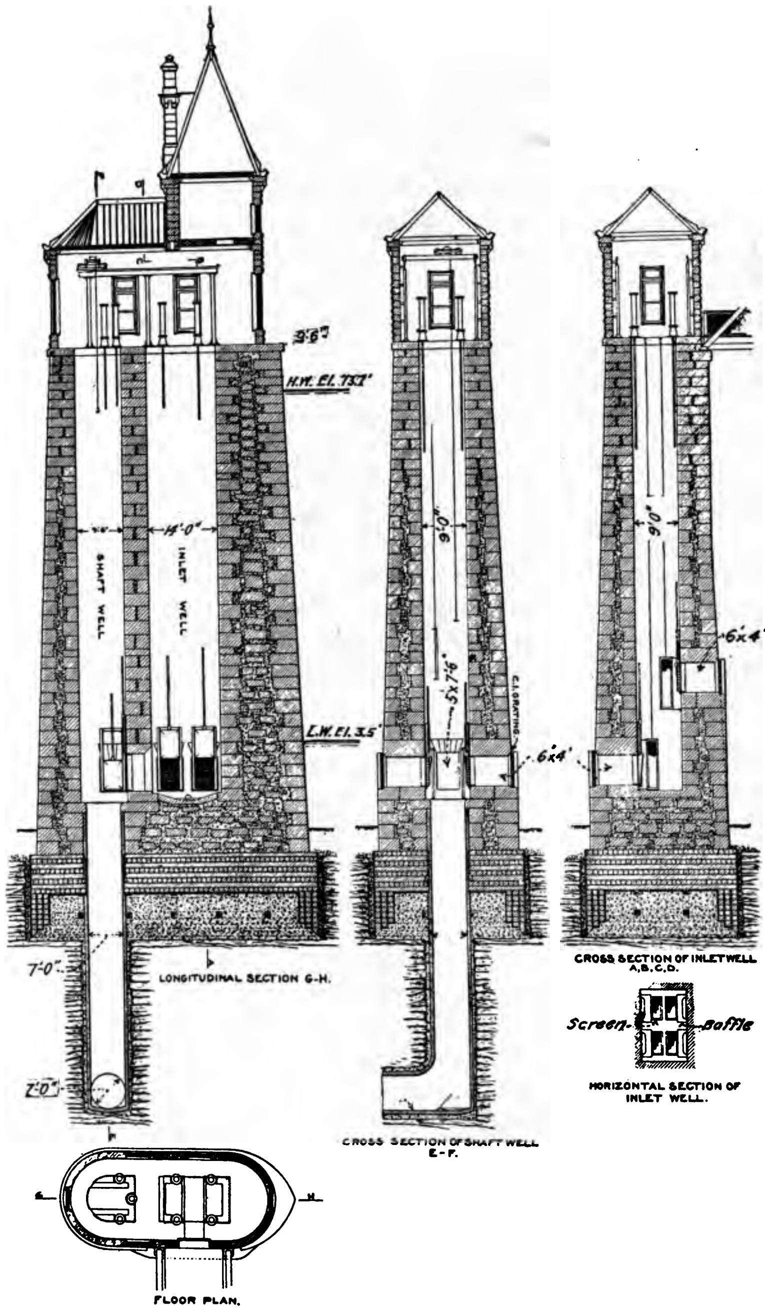


FIG. 161. — Intake-pier of Cincinnati, Ohio.
(Built 1898–1905.)

The shaft-well has two 4- by 6-ft. ports: one on the channel, and one on the shore side. A 6- by 7.5-ft. opening connects the inlet- and shaft-wells.

Each port and opening is provided with a sluice-gate and screen, with suitable guides. The gates are operated by hydraulic power, and the screens by an electric traveling hoist. The screens consist of brass wire cloth of $\frac{1}{4}$-in. mesh. Heavy horizontal cast iron grate-bars protect the outer openings of the ports. Means are provided for flushing out sediment from the inlet-well through a drain into the river.

The pier is built of the best Ohio River sand-stone, except the coping and nosing which are of granite. The masonry is laid in regular courses. The top of the pier is connected with the Kentucky shore by a steel bridge, 14 ft. wide and 320 ft. long, on which a standard gauge tract is laid. A handsome stone building with a tower on its up-stream side is built on top of the pier. It contains the plant for operating the gates and screens. The shore end of the bridge is closed by a brick building which is used as a store-room and repair-shop.

The shaft-well of the pier is extended downward to form the connection between the inlet-well and the inlet-tunnel. This part of the well, which is 7 ft. in diameter, is lined with brickwork. It makes a curved connection with the tunnel which extends under the river to the pump-pit shaft on the Ohio shore. The tunnel is 7 ft. in diameter and 1430 ft. long.

The new water works of Cincinnati, of which the inlet pier and tunnel form a part, were constructed in 1898 to 1909.

CHAPTER XIV

AQUEDUCTS

212. General Description. — An aqueduct * is an artificial channel for conveying water, such as a ditch, canal, pipe-line, or masonry conduit. In a narrower sense, the word is sometimes applied to a masonry structure built for conveying a canal over a river or hollow. Most modern aqueducts of importance include two or more of the different types of channels mentioned above. They consist principally of masonry conduits, built by the *cut-and-cover* method, or in tunnel; and cross depressions along their routes either on aqueduct bridges, or by inverted siphons. The latter may consist of a number of pipes, a reinforced masonry conduit, or a *pressure tunnel.* For small cities, the aqueduct is usually formed by one or more pipe-lines.

213. Ancient Aqueducts. — Ruins of ancient aqueducts, some in an excellent state of preservation, are to be found in Africa, Asia Minor, and in various parts of Europe. The earliest of these works consisted of earthen or stone pipes. Later, the aqueducts were built in tunnels, in order to conceal them from enemies. Valleys, however, were generally crossed by arcades. An exception to this rule occurred in the aqueduct of ancient Lyons, France, which crossed two deep valleys by siphons of lead pipes, encased in masonry.

214. *The aqueduct of Patara,* in Syria, is one of the oldest water conduits. It was formed by a canal, covered with large stone slabs, except at the crossing of a deep ravine, about 200 ft. wide, where the conduit consisted of a pipe forming a siphon, which was laid on a stone embankment. Originally, this siphon was made of earthen pipes, but later these pipes were replaced by stone blocks, each containing about 3 cu. ft., which were bored through the center to a diameter of about 13 ins. Each stone block had an annular projection at one end, and a recess, about 3 ins. deep, at the other, the blocks being thus connected by a kind of bell-and-spigot joint. The stone blocks were securely united by iron clamps, run with lead, and the joints were filled with cement. Air-vents, 7 ins. in diameter, were provided on the siphon, at intervals of about 20 ft., and were closed by stones laid in cement. This aqueduct, which is one of the finest ruins of early Greek construction, is in a very good state of preservation.

* Derived from the Latin words: *Aqua,* water, and *ducere,* to lead.

215. *The aqueduct of Laodicea,* in Asia Minor, was similar to the Patara conduit. Before it reached the city, the water was conveyed across a deep valley by a siphon of two lines of stone pipe, respectively, about 8 and 10 ins. in diameter, which were subjected to a pressure of about 60 lbs. per square inch.

216. *The aqueduct of Jerusalem* that conveyed water from the Pools of Solomon to the temple, consisted of earthen pipes, about 10 ins. in diameter, which were encased in masonry. This conduit, which was about 12 miles long, was constructed generally below the surface of the ground. The reservoirs, where the supply from springs was impounded, are still in existence, and ruins of the aqueduct are found in many places.

217. *The aqueduct of Athens* (Plate I, Fig. 1),* built about 500 B.C., was formed of stone blocks, laid dry below ground. Each side was built of one course of stone, with vertical faces. The top stones and the bottom stones were cut out to curves, so as to form, respectively, the arch and invert of the conduit.

218. *Roman Aqueducts.* — The Romans built the greatest aqueducts of ancient times. Ruins of these works are to be found in many places.

The city of Rome was supplied with water in 97 A.D. by nine aqueducts, which were fully described by Sextus Julius Frontinus,† who was Commissioner of Water Supply (curator aquarum) of Rome at that time. Details of these conduits are given in Table XLI.

Some additional aqueducts and branch conduits were built subsequent to Frontinus's time. The most important of these conduits, and the dates of their construction, are: the Trajana, 109 A.D.; the Hadriana, 120 A.D., and the Aurelia, 185 A.D. These conduits were built mainly under ground, but some valleys and the Campagna, a plain near Rome, were crossed by arcades, which sometimes carried two or more different conduits. Thus the Marcian arcade (Plate II) carried the Marcia, Tepula, and Julia aqueducts; and the Claudian arcade (Plate III) supported the Claudia and the Anio Novus conduits. The Romans built these expensive arcades because the only metal available, at that time, for pipes was lead. Pipes of this costly material (Plate I, Fig. 2) were used for distributing the water in Rome.

The finest ruins of old Roman aqueducts are: the Pont du Gard (Plate IV) near Nismes, France, and the aqueduct of Segovia in Spain. Both of these conduits were built in the first century of the Christian era.

* This view of the aqueduct was taken near the theatre of Dionysos, Athens.

† "Frontinus and the Water Supply of the City of Rome," by Clemens Herschel, Hydraulic Engineer, M. Am. Soc. C. E. "The Old Roman Aqueducts," by the Author; Proc. Municipal Engineers of the City of New York, 1903, p. 26.

PLATE I.

FIG. 1.—Aqueduct of Athens. (Built about 500 B.C.)

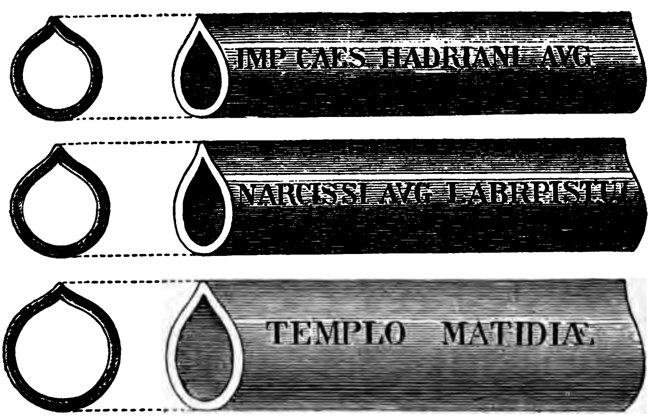


FIG. 2.—Water Pipes of Ancient Rome.

PLATE II.

Fig. 1.—Marcian Arcade, near Rome (carrying Marcia, Tepula, and Julia Aqueducts).

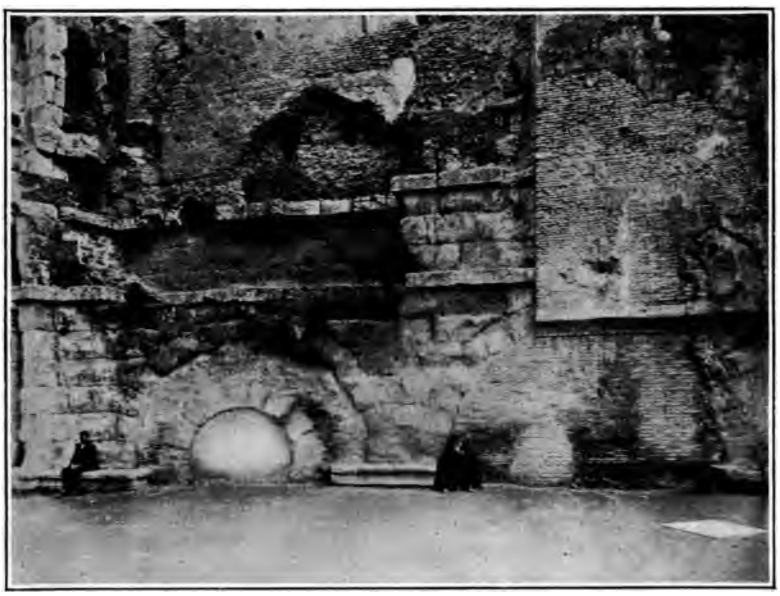


Fig. 2.—Marcia, Tepula, and Julia Aqueducts, near Rome.

TABLE XLI. — ROMAN AQUEDUCTS, DESCRIBED BY FRONTINUS, 97 A.D.*

Name	Date	Length in miles		Total	Level of invert in Rome above sea-level	No. of castellae
		Below ground	Above ground			
Appia	312 B.C.	10.23	0.06	10.29	65.60	20
Anio Vetus	272–269 B.C.	39.33	0.20	39.53	157.44	35
Marcia	144–140 B.C.	49.87	6.86	56.73	192.31	51
Tepula	125 B.C.	5.51	6.44	11.95	198.87	14
Julia	33 B.C.	7.75	6.44	14.19	209.03	17
Virgo	19 B.C.	11.83	1.14	12.97	65.60	18
Alsietinus	10 B.C.	19.96	0.33	20.29	54.12	....
Claudia	38–52 A.D.	33.31	9.35	42.66	221.07	92
Anio Novus	38–52 A.D.	45.32	8.64	53.96	230.91	....
Total	....	223.11	39.46	262.57	....	247

* The data given in this table have been taken from "Aqueducts of Ancient Rome," by John H. Parker, London, 1876; and "The Ruins and Excavations of Ancient Rome," by R. Lanciani, New York, 1897.

The Pont du Gard, which has a maximum height of 160 ft. above the river, is still in an excellent state of preservation. It is composed of three tiers of semicircular arches. The lowest tier contains six arches: one of 80 ft. 5 ins. span, three of 63 ft. span, and two of 51 ft. span. The second tier contains eleven arches, each of 51 ft. span. In the third tier there are thirty-five arches, each of about 16 ft. span. The widths of the arcade at the first, second, and third tiers are, respectively, 21, 16, and 10 ft. Some of the arches at each end of the structures have been destroyed. The present length at the molding terminating the second tier is 883 ft. This structure and all of the aqueducts mentioned in Table XLI were built entirely of dry masonry, cement-mortar being only used for lining the water channel. At a later period, the aqueducts were built of concrete or rubble masonry.

The aqueduct of Segovia, Spain, which has a maximum height of 102 ft., is still in service.

219. During the Middle Ages no aqueducts of importance were built. The communities obtained their water supplies from springs, shallow wells, rivers or lakes, and distributed the water in pipes of lead, or in bored logs.

London obtained from Tyburn, in 1236, a supply of spring water, which was conveyed in a lead pipe. In 1613 this city constructed its first aqueduct, to bring water from two springs in Hertfordshire to London. This conduit consisted simply of an open ditch, about 38 miles long. It was called *The New River*, and, with improvements in its alignment and construction, is still one of the aqueducts that supply London.

Paris built its first water works about 1183. They consisted of a reservoir for spring-water, from which the water was conveyed to the city by a lead pipe, which was still in service in 1878.

220. Modern aqueducts have been built during the past century for many large cities. Short descriptions of some of these conduits, and, also, of smaller aqueducts, consisting only of one or more pipe-lines, are given on pp. 258 to 342. We shall discuss some of the important details connected with the design and construction of aqueducts in the following pages.

221. Masonry Conduits. — Modern aqueducts, supplying large cities, consist mainly of masonry conduits, constructed either by the cut-and-cover method, or in tunnel.

In designing such a conduit, the data given are usually the maximum quantity of water which is to be conveyed from the source of supply to the place of consumption, and the available head. Based upon these data, the cross-section of the conduit is designed, according to theoretical and practical considerations. On a long aqueduct, different types of cross-sections may have to be adopted for different sections of the conduit.

222. *The cross-section of the conduit* will depend upon whether the conduit is to be subjected to pressure, or to discharge water by gravity. In the former case, the cross-section of the conduit is almost invariably made circular, so as to have equal strength in all directions. In the latter case, however, a great many different types of cross-sections may be adopted.

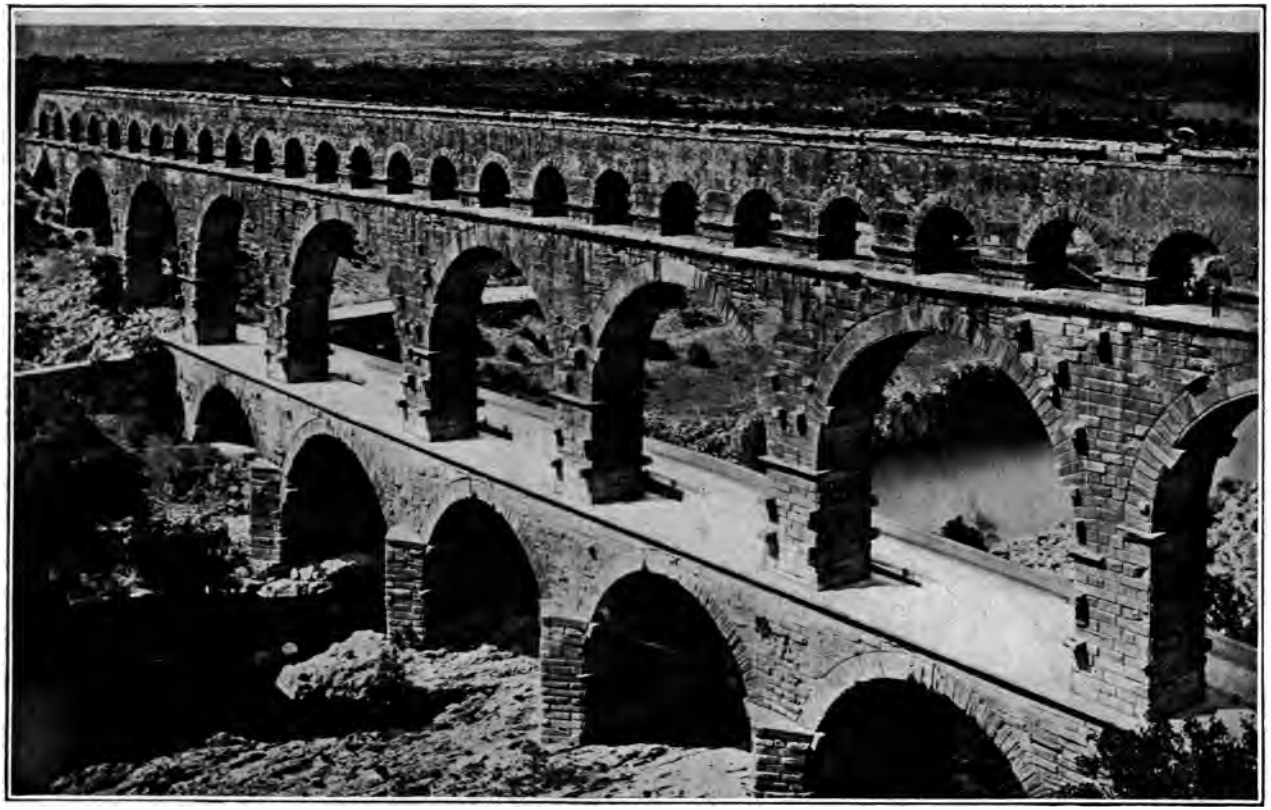
Theoretically, the best cross-section for a conduit is always a circle, as this form gives the greatest hydraulic radius for a given area of cross-section and, consequently, the greatest velocity of flow. There are, however, practical considerations which often lead to a modification of the circular section, and a *horse-shoe section* has, generally, been adopted for large modern conduits. Compared with the circular form, it has the following advantages: (1) it has more head-room for a given area; (2) it is more easily constructed, as the side walls can be built, before the invert is begun; (3) the construction tracks may be left undisturbed until the end of the construction, when they are taken up during the laying of the invert masonry.

The greatest velocity of flow with a horse-shoe section of a given area will be obtained when the height is about equal to the width. On embankments, however, the width may have to be made greater than the height, in order to obtain a sufficient base for the conduit, to prevent settling.

PLATE III.

Claudia and Anio Novus Aqueducts, near Rome.

PLATE IV.



Pont du Gard, in Southern France. (Built in First Century of Christian Era.)

When a large *gravity conduit* is constructed in a tunnel, it will be advantageous to make the height of the cross-section greater than its width, as this reduces the quantity of heading excavation, which is much more expensive than the excavation of the bench, and, also, reduces the span of the arch of the roof. Oval cross-sections have been adopted for small conduits, and, in some cases, a foot walk has been constructed on one or both sides of an aqueduct, for convenience during inspections or repairs. When a circular section is adopted for a conduit through which water is discharged by gravity, the greatest discharge will occur theoretically, when the water subtends an arc of 51° 48′ (p. 33).

223. Construction of Masonry Conduits. — Prior to 1900, most masonry conduits were built of brickwork. Since this date conduits have been generally constructed of concrete, as it is cheaper than brickwork, well adapted for difficult forms of construction, and water-tight under ordinary pressures, if properly mixed and laid. Voids that may occur in brickwork and rubble masonry are practically impossible with concrete.

One of the troubles experienced with concrete is cracks, due to changes of temperature, which generally occur at places where the laying of the concrete was interrupted sufficiently long to permit the cement to take its initial set. These cracks may be prevented by placing expansion joints in the masonry, at intervals not exceeding about 60 to 75 ft. If they are placed farther apart, expansion cracks are apt to occur between them. In a concrete conduit, a complete transverse expansion joint should be placed at the intervals mentioned, tar, asphalt, paper, etc., being used in the joints, in order to permit the masonry to move slightly. This method of construction prevents, also, cracks due to settling. An objection, however, to expansion joints is the fact that they are apt to permit leakage from the aqueduct.

In the Catskill aqueduct (p. 290), tongue-and-groove expansion joints were used exclusively at first. Although these joints gave good results, when carefully made, there were practical difficulties in the way of getting them smooth enough and true enough, in the hurry in which the work had to be carried on. Leakage was found to be apt to occur through the invert joints, and steel bands, about $\frac{3}{8}$ in. thick by 6 ins. wide, dipped or painted with asphalt, were placed as water-stops in the joints between day's work. Strips of copper, $\frac{1}{32}$ in. thick, and lead were, also, tried. Half of the width of the steel was buried in the concrete first cast. When leakage was found to occur at such joints, it was stopped by grouting.* In the concrete aqueduct built recently for Los Angeles, Calif. (p. 319), no expansion joints were used in the masonry, reliance being

* "Waterworks Handbook," by Flinn, Weston, and Bogert, p. 274.

placed upon its being strong enough to resist the stresses caused by changes of temperature.

Where changes of cross-section occur in a masonry conduit — such as the transition from a horse-shoe gravity conduit to a circular pressure tunnel — they must be made gradually and smoothly, so as to reduce the loss of head at such points to a minimum. In a gravity conduit built in trench or tunnel, ground water may force its way into the conduit and injure the masonry. If the ground water is of good quality, small openings, called *weepers*, should be left in the masonry at intervals of about 20 ft., to permit the ground water to enter the conduit. The weepers on the two sides of the conduit should be staggered.

Masonry conduits are sometimes placed on earth fills, but in such cases the embankment must be made in thin layers, about 3 ins. thick, thoroughly wetted and rolled. At such places the bottom of the conduit should be increased in width, so as to reduce the pressure in the foundation. A section of the Sudbury aqueduct, which was built in this manner on an embankment 34 ft. high, has not settled appreciably.

224. *Forms.* — Until recently, the forms used for laying the masonry in a conduit were made of wood, in the usual way. In works of this kind built of late years, collapsible steel forms made by the Blaw Steel Construction Co. of Pittsburgh, Pa., the Ransome Concrete Machinery Co., etc., were used with great success. These forms should be hinged at the crown, and arranged in such a manner that they may be removed progressively from the bottoms of the side walls to the crown. When the forms are taken out, the concrete is apt to peel off at the crown. This difficulty can be overcome by greasing the forms with a suitable lubricant, such as crude vaseline or oil, etc., before the last concrete is placed, and by using care in removing the forms. Leakage through the forms must be prevented, as the concrete will otherwise be apt to be porous at the bottom of the side wall.

225. Distribution of Available Head. — In an aqueduct composed of different kinds of channels, such as masonry conduits, pipe-siphons, unlined tunnels, etc., the *available head* from the source of supply to the place of discharge is not distributed equally, but according to the requirements of the different parts. The capacity of the aqueduct being given, the grades of the different parts must be based upon the minimum available head. As the loss of head is greater in the pipe-siphons, unlined tunnels, etc., than in the smooth masonry conduits, more head must be allowed for them. Curves in the alignment will not cause much loss of head, but appreciable loss will occur at changes of cross-section. The final subdivision of the available head can only be arrived at by trial calculations.

226. Grades. — If the velocity of the water in a conduit is less than about 2 ft. per second, sand contained in the water will be deposited, and reduce the area of the conduit. If, on the other hand, the velocity should exceed about 7 ft. per second, the sand and stones that may be washed through the conduit by the water are liable to injure the masonry, unless it is built of very hard stone. The velocity of the water in a conduit should, therefore, be between 2 to 7 ft. per second, about 3 to 4 ft. per second being a very good average velocity, which will usually be obtained in large aqueducts having a grade of 9 to 12 ins. per mile.

227. Gate-houses. — Every aqueduct must be occasionally emptied for inspection, cleaning or repairs. Some provision must, therefore, be made for closing the inlet and outlet, and for discharging the water from the conduit. For these purposes gate-houses, provided with suitable sluice-gates, are constructed at the inlet, outlet, and one or more intermediate points. The general arrangement of these structures is described below.

228. *The inlet gate-house* consists of a substructure containing the water-chambers, sluice-gates, etc., and of a suitable superstructure for protecting the hoisting machinery of the gates, stop-planks, etc. As every mechanical contrivance is liable to get out of order, it is advisable to have two sets of inlet sluice-gates and, in addition, two sets of stop-planks, by means of which a coffer-dam can be quickly formed in front of the outer set of gates. To reduce the length of the stop-planks, a wide substructure is divided by a partition-wall into two divisions. There are two cross-walls in which the openings for the sluice-gates are constructed. Two sets of grooves are cut in the side-walls for the stop-planks. One set is ordinarily used for screens, to keep fish and floating objects from entering the conduit. For a small conduit the substructure requires only one division.

The flow in the conduit is ordinarily regulated by means of the inner set of sluice-gates. If the level of the reservoir is much above the soffit of the conduit, several sets of gates are required, to reduce the head of the water so that it shall flow through the conduit without filling it entirely. The different sets of gates are placed in cross-walls which divide the water chamber into a number of small chambers. The gates nearest the reservoir are opened a trifle, the next set a little more, and so on, the height of the water in the conduit being thus regulated. A good illustration of such a case is the large inlet gate-house of the New Croton aqueduct (p. 277).

229. *Outlet Gate-house.* — An aqueduct discharges the water it conveys either into a service-reservoir, or into a large terminal gate-chamber from which the water is distributed by pipes. A description of such a gate-house, built for the New Croton aqueduct, is given on p. 284.

When a conduit discharges into a reservoir that is divided into two basins — as is generally the case — the conduit should terminate at the division-wall of the reservoir, and a small gate-house, provided with sluice-gates, stop-planks, etc., arranged so that water can be discharged into either basin, should be built at the end of the conduit.

230. *Blow-offs and Waste-weirs.* — At suitable points on the line of the conduit — about 5 to 10 miles apart — *blow-off gates* are provided for emptying the conduit for inspection, cleaning or repairs. These gates are generally placed in special gate-houses, located near streams into which they discharge the water from the conduit. Waste-weirs are usually constructed in the gate-houses, to regulate the maximum height to which the water can rise in the conduit. Their height may be varied by means of stop-planks.

The substructure of the gate-house is divided by a partition wall, built parallel with the center-line of the conduit, into two chambers: one forming part of the conduit, and the other receiving the waste-water from the blow-off gates or waste-weir. The latter is connected by a culvert or a paved open ditch with some water course (p. 287).

By constructing blow-off gate-houses at different points on the line, an aqueduct is divided into sections, any one of which can be emptied, independently of the others, for inspection, cleaning or repairs.

231. Screens are required at the inlet of aqueducts conveying surface water. They are usually placed in the inlet gate-house. Where the water enters the gate-house, racks of metal or wood pieces, set edgewise, 4 to 6 ins. apart, are placed, to exclude large objects from the gate-house. In an inner screen-chamber, coarse copper screens with meshes of 1 to 1½ inches are set, and at the entrances to the aqueduct, pump-wells or filters, finer screens having about 4 to 6 meshes per inch are provided.

The screens are usually made of copper or bronze wire, about No. 14 to No. 16 Birmingham wire gauge, which is fastened by means of staples to wooden frames. The screens should be placed vertically in grooves, and two sets should be provided, so that one can be in service while the other is raised to be cleaned. A recess should be constructed in front of the screens for receiving the trash which may drop from the screen, and means for cleaning out this recess, occasionally, should be provided.

The standard screen of the Boston water works* is made of No. 16 (Birmingham wire gauge) copper, with 6 meshes per inch. The frames are made of white pine and fastened with tree-nails. The netting is

* "Waterworks Handbook," by Flinn, Weston, and Bogert, p. 293.

stapled to the up-stream side of the frame, and a ½-in. board, of the width of the frame, is placed over the staples. A *dirt catcher* — a wire basket of the same netting as the screen — is fastened to the bottom of the frame.

232. Stop-planks are used for forming temporary dams across aqueducts, or in front of sluice-gates, etc., when repairs are needed. They are placed in grooves, cut in the masonry, or made of metal. Fig. 162 shows a stop-plank made out of 6 × 12-in. yellow pine. For convenience, several stop-planks may be bolted together and weighted by a piece of I-beam, etc. This facilitates the lowering of the stop-planks.

The stop-planks used at the Hinckley Dam* of the N. Y. State Barge Canal are made of white oak timbers, fastened between I-beams, and bolted together in convenient sections. Bronze rollers are provided

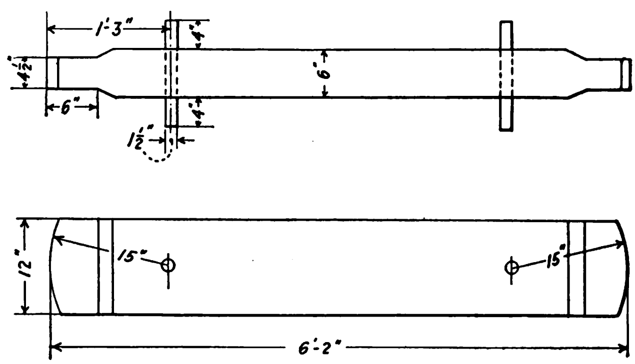


FIG. 162. — Stop-plank.

for each section, to reduce the friction in both directions. Instead of stop-planks, steel shutters may be used for great depths of water. Their bearing strips may be of wood.

233. Culverts. — In constructing a masonry conduit near the surface of the ground, the brooks and streams that are encountered must be carried under or over the conduit by means of culverts or open water-channels. If a brook is liable to carry driftwood at times, it should be made to flow over the conduit, if practicable.

234. Ventilators are needed for a conduit at intervals of about 1 to 2 miles. They permit the air to escape while the conduit is being filled, and to flow in when the conduit is being emptied. They maintain, also, a circulation of air over the water, and prevent foul air from collecting in the conduit and contaminating the water. The ventilators may be constructed as points of admittance to the conduit. The blow-off gate-

* *Eng. News*, Feb. 15, 1912, p. 282.

houses serve as ventilators at points at which they are constructed. Fig. 163 shows a ventilator constructed for the Old Croton aqueduct.

235. Measuring the Flow through an Aqueduct. — This is generally done with a current-meter. The gauging may be done at gate-houses, or at special shafts or chambers, constructed for this purpose. Fig. 164 shows such a shaft on the New Croton aqueduct of New York. The discharge through the aqueduct is determined at this point with a current-meter as follows:

The area of the water-channel is subdivided into a great many small areas of equal size, each containing about a square foot. The average velocity in the conduit is found in two ways: first, by moving the meter

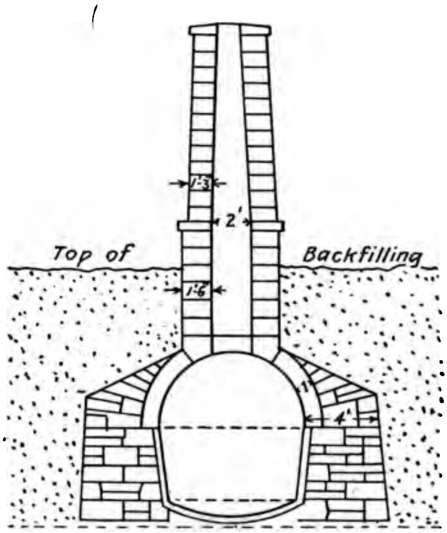


FIG. 163. — Ventilator Shaft of Old Croton Aqueduct.

slowly and uniformly through all the small areas, recording the time and the total number of revolutions; second, by leaving the instrument for a minute in each separate area, recording the time and the total number of revolutions. From these observations, which are plotted as shown in Fig. 164*a*, the average velocity of the whole area is computed. The results obtained by both methods agree very closely.

The current-meter is attached to a brass rod ($1\frac{1}{16}$ in. in diameter) which is passed through a swivel. The rod is made in five-foot sections which are screwed together. It is provided with a number of projecting lugs which fit accurately into grooves of a brass limb that is placed in the manhole and fastened to three 4- by 8-in. yellow pine posts. The sections in the limb are $\frac{7}{16}$ in. deep, and are cut $\frac{3}{4}$ in. from center to

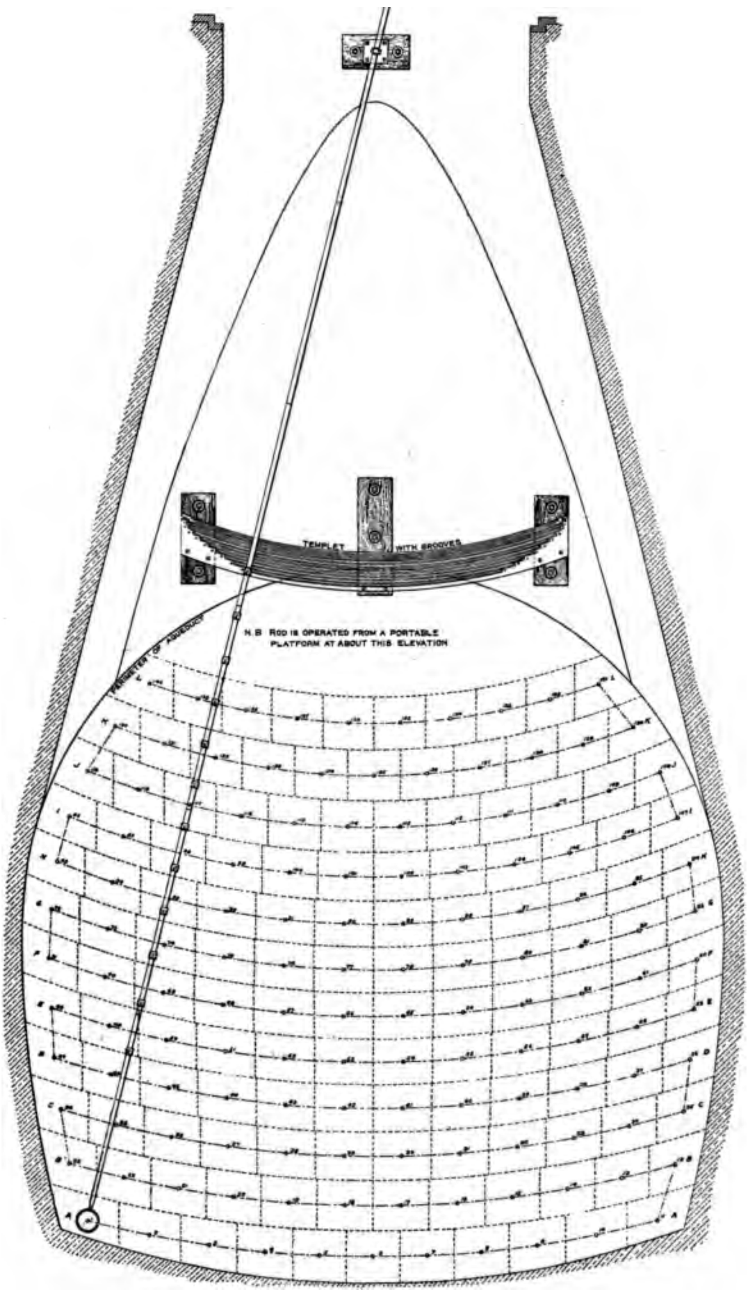


FIG. 164. — Gauging Shaft of New Croton Aqueduct.

center. The lugs on the rod and the grooves of the limb are marked in such a manner that the position of the current-meter in the aqueduct is always known.

Four men are required for making the observations. The first moves the current-meter by shifting the lug of the rod along the grooves of the limb; the second holds the meter against the current by means of a stout rod, fastened by wire to the brass tube, just above the water; the third is stationed at the top of the shaft, and assists in raising or lowering the rods; the fourth, watch in hand, times the observations and gives the signals.

236. The Venturi meter, described on page 530, has been used extensively for measuring the flow through pipe-lines. Three meters

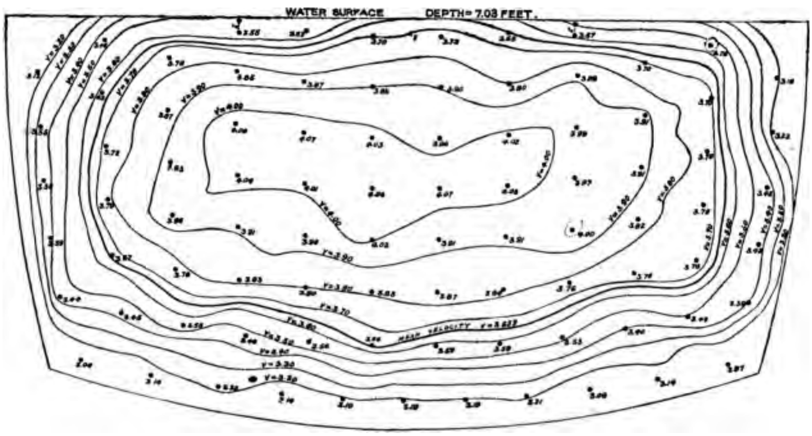


FIG. 164*a*. — Velocity Contours in New Croton Aqueduct.

of this type — the largest thus far made — have been constructed in the Catskill aqueduct (p. 312).

237. Aqueduct bridges are one of the means by which a conduit can be carried across a valley or a depression. They are usually constructed of masonry.

238. *The Cabin John bridge,** built in 1863, carries the Washington aqueduct, which is 9 ft. in diameter, across Cabin John Run. It is a fine stone arch bridge of 220 ft. span. Originally the depth of the water in the conduit on the bridge was only to be 7 ft., but it has been increased to 9 ft. Since this was done, there has been more or less trouble with leakage.

239. *High Bridge* (Plate V), built in 1840–48, is a fine structure which carries the Old Croton aqueduct across the Harlem River to

* *Eng. Record,* April 1, 1911, p. 356; and *Eng. News,* July 10, 1913, p. 67.

Manhattan Island, New York. It is 1450 ft. long between the gate-houses at its ends. The bridge has fifteen semicircular arches, eight having spans of 80 ft., the other spans being 50 ft. The soffit of the arches at the crown is 100 ft. above high-water. The water is carried across the bridge in two 36-in. cast iron mains and in a wrought iron pipe, 90½ ins. in diameter, which are placed in a vault which was constructed on top of the bridge. Provision for contraction and expansion was made for the large pipe by placing stuffing-boxes at its ends.

240. *Aqueduct bridges for the Sudbury aqueduct of Boston** were built in 1873–1879 across Waban Brook and across the Charles River. The first of these bridges is 536 ft. long and has nine semicircular stone-arches with spans of 44 ft. 8 ins. The outer spandrel-walls are built of rubble masonry, faced with ashlar. They are 4 ft. thick, and are capped for the whole length of the bridge by a string-course forming the cornices. Two interior brick spandrel-walls, covered with flagstones, support the bottom of the masonry conduit. The longitudinal space left between the spandrel-walls above the arches are spanned by continuous brick arches, forming, with the spandrel-walls, three galleries extending under the conduit for the whole length of the bridge. Leakage from the aqueduct finds its way into one of these galleries, and is discharged by outlet pipes, provided at the piers and abutments. Access to the galleries is obtained by means of manholes, and small openings are provided to give light and ventilation.

The aqueduct bridge across the Charles River has seven stone arches, the main arch having a clear span of 130 ft.

241. *The Assabet bridge*, on the Wachusett aqueduct of Boston, has a span of 29.5 ft. In the Sudbury bridges, considerable leakage from the aqueduct occurred. This trouble was obviated in the Assabet bridge by lining the aqueduct with lead, which has made the conduit water-tight. The lining was made of sheets of lead, 9 by 16 ft. in size, weighing 5 lbs. per square foot. The sheets were burned together without solder, and the bottom of the aqueduct, against which the sheets of lead were placed, was plastered with a ½-in. coating of cement mortar, coated with asphalt. A lead lining, made of lead sheets $\frac{1}{16}$ to $\frac{1}{8}$ in. thick, was placed later on the wetted perimeter of the Sudbury bridges.†

242. Inverted siphons, called briefly *siphons*, are frequently used to carry an aqueduct across a valley or depression. They may consist of one or more lines of iron, steel, concrete or wooden pipe, or of a pressure-tunnel.

* Report on the Construction of the Sudbury Aqueduct made by A. Fteley, Resident Engineer, to the City Engineer of Boston, 1882.

† *Eng. Record*, March 25, 1911, p. 328.

243. Pressure-tunnels, driven through solid rock, and having a sufficient covering of sound rock, have been used as siphons, in connection with modern aqueducts. The New Croton aqueduct of New York was built across the Harlem River as a pressure-tunnel, 10.5 ft. in diameter and about 1250 ft. long, at a depth of about 300 ft. below high-water in the river.

244. *On the Catskill aqueduct* of New York, several deep valleys are crossed by pressure-tunnels, the most important one being the tunnel, 14 ft. in diameter and 3022 ft. long, which was driven under the Hudson River, about 1100 ft. below high-water. What is known as the City tunnel of the Catskill aqueduct is a pressure-tunnel, 18 miles long, with a diameter reduced gradually from 15 to 11 ft., driven at a depth of 200 to 750 ft. below the surface.

Pressure-tunnels should be given a circular section and a lining strong enough to withstand the maximum stresses which may occur, with the tunnel full or empty, and which must be made as water-tight as possible by grouting. All seams and fissures in the rock near the tunnel should be filled with grout, forced into the seams by pressure.

In the construction of the Harlem River siphon, grout, under a pressure of 50 lbs. per square inch, was forced by means of an ordinary ship pump into the voids in the masonry and the seams of the rock, after the masonry lining had been laid. In the Catskill siphons, great improvements in grouting machines were made, and, in some cases, the pressure under which the grouting was done was as high as 300 lbs. per square inch.*

245. *Wachusett Aqueduct.* — On a two-mile stretch which might be under pressure, cut-off walls of masonry were built over the arch at intervals of 50 ft., and the spaces between these walls were filled with dry stone packing, and afterwards grouted under a pressure of 50 lbs. per square inch.

246. *Failure of a Pressure-tunnel.* — The Los Angeles aqueduct was originally constructed across the *Sand Canyon* as two inclined pressure-tunnels, 9 ft. in diameter, connected across the narrow bottom of the gorge by an 8½-ft. steel pipe. The rock through which the tunnels were driven was hard gray granite, massive in appearance. The maximum head on the siphon was 455 ft. On the north side, the incline joined a covered conduit, and, on the south side, the incline was connected with a rock tunnel. The total length of the siphon was 2787 ft., of which length 1517 ft. consisted of a horizontal tunnel and steel pipe. The covering over the north and the south ends of the pipe, where it joined the tunnels, was, respectively, 83 and 170 ft. The tunnels were lined with 1 ft. of

* "Waterworks Handbook," by Flinn, Weston, and Bogert, p. 285.

1–2–4 concrete, and this lining was carried forward for ten feet over the ends of the pipe. Holes were left in the concrete lining for grouting all the horizontal portions of the tunnel, and for some 50 ft. up the incline on the south side. The grout was forced into the masonry with a triplex pump, under a pressure of over 200 lbs. per square inch. The concrete was discharged through an 8-in. pipe, behind the forms, from the head of the incline, and flowed into all crevices and spaces in the rock. No grout was pumped at other parts of the tunnels behind the concrete, on account of the very compact manner in which the concrete was delivered.

When the siphon was first tested by hydrostatic pressure, some leakage was noticed, and a break finally occurred on the side-hill, approximately 80 ft. in elevation above the top of the pipes and about 150 ft. west of the pipe. The blow-out was immediately opposite the north end of the steel. An inspection made, after the pipe was emptied, showed that a large crack, about 3 ins. wide, had been made in the siphon. It started at the bottom of the steel, on the north side, and ran diagonally up each side to the spring line, at a distance of about 50 ft. from the end of the steel. The crack then followed the spring-line, with an opening of about 1 in. in width, for a distance of about 100 ft. The crack stopped in the top of the tunnel, 230 ft. from the end of the steel. No indication of failures occurred at the south end of the pipe.

An attempt was made to repair the siphon by placing a 66-in. steel pipe inside of the original 100-in. pipe, and by filling the space between the pipes solidly with concrete and grout. After the repairs had been made, the siphon was tested again. This time, leaks quickly appeared, about two-thirds of the way up the hill on the south side. The leakage which was at first largely absorbed by the rock and soil covering, finally caused the rock and soil to slip, resulting in a great cutting of the hill on the south side. About 250,000 cubic yards of material was displaced in this manner, and moved from 30 to 50 ft. About 20,000 cubic yards of this material reached the bed of the canyon.

The damage caused by this failure — the only one that occurred on the entire 233 miles of aqueduct constructed — was so great, that the siphon had to be abandoned and replaced by an all-steel siphon on a new location across the canyon. The $8\frac{1}{2}$-ft. steel pipe in the original siphon was placed in the new siphon.

247. Siphons of the Los Angeles Aqueduct.* — There are twenty-three inverted siphons, aggregating in length 11.4 miles, on the line of the Los Angeles aqueduct. Eight of these siphons, comprising a length

* "Complete Report on Construction of the Los Angeles Aqueduct," published by the Department of Public Service, 1916.

of 2.7 miles, are reinforced concrete pipes, 10 ft. in diameter, which are subjected to heads varying from 40 to 75 ft. The remaining siphons consist of riveted steel pipes, 7.5 to 11.5 ft. in diameter, having a thickness of metal of $\frac{1}{4}$ to $1\frac{1}{8}$ ins. The steel plates for these siphons were made by eastern manufacturers, in accordance with the specifications for boiler plate steel of the American Society for Testing Materials which were adopted for the Los Angeles aqueduct. The minimum thickness of plate was fixed at $\frac{1}{4}$ in. and was used for 10-ft. pipes for heads up to 144 ft. Plates $\frac{1}{2}$ in. or less in thickness had lap-joints with edges sheared for outside caulking, and were rolled and punched at the shops, all rivets being driven in the field. For all plates over $\frac{1}{2}$ in. in thickness, triple-riveted butt-joints, with edges planed, were used. The plates were riveted together in two-ring sections, except the heavy plates across the bottom of canyons, which were shipped in longer sections.

The best results were obtained by laying the pipes in shallow trenches that support about one-third of the bottom circumference of the pipes. This can be successfully done on gentle slopes where the soil conditions are favorable.

248. *Piers.* — Most of the pipes were laid on concrete piers 1 to 2 ft. wide. The best distance between centers to piers for a 10-ft. pipe with $\frac{1}{4}$-in. shell was found by experiment to be about 24 ft. A case happened in the field where two piers on a side-hill settled away from a pipe, leaving the supports 72 ft. apart. The empty pipe, which was 11 ft. in diameter and $\frac{1}{4}$ in. thick, stood suspended without injury, until the defective supports could be replaced.

At first, the piers were built up the sides of the pipes to the horizontal diameter. It was found, however, that the horizontal diameter of some of the pipes was as much as 9 ins. greater than the vertical diameter, when the pipes were empty. When they were about two-thirds full, the distortion became greater, and the leverage it exerted on the sides of the piers was sufficient to rupture them on lines radial to the pipe. The sides of a number of the first piers, which were 1 ft. wide at the top, 2 ft. thick, and reinforced with six $\frac{5}{8}$-in. round rods, were broken off in this manner. The piers were carried up high on the sides, in order to stiffen the plates on the sides of the pipe, but it was noticed that when the pipe assumed its circular form, after being filled with water, that it did not bear for over 3 to 4 ft. of its circumference on the ruptured pier. Based upon this experience, it was decided to give the piers a larger section, but not to carry them up so high. The piers were, therefore, built up to the bottom of the pipes, and the reinforcing rods were left projecting. After the pipes had been filled with water, the piers were completed.

249. *Field Riveting.* — The practice was to rivet two rings or more together, at the side of the trench, and then to roll the two-ring section, 12 ft. in length, into the trench. Under favorable conditions, a crew, consisting of: 1 riveter, 1 heater, 1 bucker, 1 sticker, would drive, under a bonus system, from 1000 to 1600 rivets, $\frac{5}{8}$ in. in diameter, in 8 hours.

In the deepest siphon, $1\frac{1}{4}$-in. rivets, each weighing about 5 lbs., were driven by means of a Thor and Boyer No. 90 air hammer, using air under 115 lbs. pressure per square inch. The air was supplied by an Ingersoll-Rand compressor, driven by a 100 horsepower motor. A No. 60 Boyer hammer was used on the end of the air-bucker inside of the pipe. The cost of driving these large rivets was 27 cents each, the energy costing 1.7 cents per kilowatt hour.

250. *Erection.* — Aerial cable-ways, or inclined railways, were used for delivering the plates at the proper points, the latter proving to be more satisfactory than the former for heavy pipe. The heaviest pipe section was for the Jawbone siphon. It weighed 52,000 lbs. and was 36 ft. 10 ins. long. This section was shipped from the East, built up, and was hauled into place by 52 mules, 6 in a row.

251. *Painting.* — The paint used for the steel pipes was a residual hydrocarbon oil, resulting from the manufacture of gas from California asphalt oil, which penetrates rust and rust scales on the metal. This paint is very cheap. It costs $4 per barrel of 50 gals. One gallon of paint was sufficient for covering 400 sq. ft. with one coat. According to circumstances, it costs from $\frac{1}{4}$ to $1\frac{1}{4}$ cents per square foot for two coats of paint.

252. *Expansion and Contraction.* — Before the pipes were filled with water, they expanded and contracted longitudinally, when changes of temperature occurred. This motion caused some injury to the piers, which was prevented later by placing a thin sheet of metal on top of each pier. In one of the siphons, 1435 ft. long, which was laid across a canyon having slopes of approximately 1 ft. vertical to 3 ft. horizontal, a change of temperature of 58 to 92° F. caused an expansion of $3\frac{1}{4}$ ins. at the north end, and of $1\frac{1}{16}$ ins. at the south end, which had the steeper slope. There was, also, in this siphon, a lateral movement of $\frac{1}{2}$ in. between morning and evening, before the pipe was filled.

In the 15,600-ft. siphon across the Antelope valley, which is the longest on the line of the aqueduct, an expansion of 23 ins. was observed from 5 A.M., the coolest part of the day, until mid-day.

No expansion joints were placed in any of the steel pipes of the aqueduct, but the pipes are securely anchored at both ends, and at horizontal angles.

The experience with these 10 miles of steel siphon-pipes on the aqueduct showed that when the pipes are placed on the top of the ground, no trouble from buckling occurs — not even when the pipes are empty — except at horizontal angles, which should be avoided.

At a horizontal angle in the Jawbone siphon, the pipe buckled during construction, so as to be depressed as much as a foot from a true circle, but when the pipe was filled with water, it returned to its circular form, as happened in all other similar cases. The head on the pipe at this point was about 100 ft.

Owing to the tendency of the pipes to move longitudinally at horizontal angles, the pipe was securely anchored at such points to a block of masonry, or completely covered with earth. No trouble from expansion and contraction occurred at vertical angles.

253. *Transition Joints.* — Steel pipes were only used for heads exceeding about 75 ft. For lower heads, pipes made of reinforced concrete were found to be cheaper. The connections between the steel pipes and the concrete pipes, under heads of about 75 ft., were difficult to make. At one place where a 10-ft. concrete pipe is connected with a 11-ft. steel pipe, under a pressure of 70 ft. head, an expansion joint was made by letting the concrete overlap the steel for 6 ft. — the width of one plate — and the annular opening of $\frac{3}{4}$ in. between the steel and concrete was caulked with oakum. It was found difficult to make this joint tight. Threaded steel rods were cast in the concrete so as to project about 6 ins.; a circular piece of angle iron was then fitted over the steel and holes were bored in it, to fit the threaded rods. The annular opening between the steel and the concrete was then caulked with oakum, and the angle was drawn down, by means of nuts, against the oakum. By repeated recaulking under pressure, the joint was finally made tight.

This type of joint was later superseded by a rigid joint, made by first riveting angle irons to the steel pipe, and then casting a large block of concrete to envelop the steel pipe. This type of rigid rod proved to be very satisfactory.

254. The Jawbone siphon* of the Los Angeles aqueduct, which is 7096 ft. long, is under a maximum head of 850 ft. The diameters of this pipe vary from 10 ft. at the ends to $7\frac{1}{2}$ ft. at the center, and the thickness of the plates ranges from $\frac{1}{4}$ to $1\frac{1}{8}$ ins. By varying the diameter of the pipe, a considerable saving in material was effected.

This pipe is laid on concrete piers, 2 ft. thick, which support one-quarter of the circumference of the pipe. The piers are spaced 36 ft. between centers across the bottom of the canyon, and closer on the hill-sides. They are founded on bed-rock on the hill-sides, and are carried

* *Eng. Record,* Dec. 20, 1913, p. 683.

down, in the center of the valley, about 8 ft. into cemented gravel. The erection of the pipe was begun in the bottom of the canyon, and was extended up each side simultaneously. The ends of the pipes are rigidly held by massive concrete anchorages at the transition sections from the adjoining conduit. Angles were riveted around the outside of the end plates, and the concrete anchorage was cast over the pipe for a distance of about 10 ft. This concrete was placed after the pipe had been filled with water, and during cool weather, the effort being made to make the casting, when the pipe was in a condition of minimum length. These joints have stood very well, and no rupture of the concrete has occurred.

CHAPTER XV

DESCRIPTIONS OF AQUEDUCTS

255. Glasgow, Scotland, constructed, in 1856–60, an aqueduct* from Loch Katrine in the Perthshire Highlands to the city, a distance of about 33 miles. A masonry dam, continued by a waste-wier, 100 ft. long, was built across the outlet of the loch, to raise its surface 4 ft. The reservoir thus formed had an available storage capacity of about 2,600,000 Imp. gals., and a drainage area of 36 square miles. Owing to the great depth of the loch — which is about 475 ft. in its deepest part — the temperature of the water remains nearly constant, varying only from about 38 to 58 degrees.

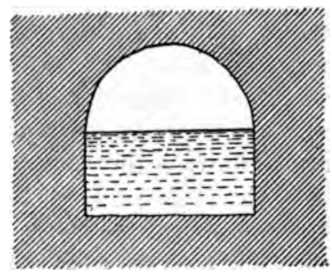
The aqueduct begins at an inlet gate-house, 40 by 55 ft. in plan, built at the loch, and provided with fish screens and with three 4- by 4-ft. sluice-gates. It has a length of $25\frac{3}{4}$ miles, and terminates at the Mugdock service-reservoir, from which the water is conveyed to the city by two lines of 42-in. cast iron pipes, each 8 miles long. Between the inlet and the reservoirs, the aqueduct is composed of $11\frac{3}{4}$ miles of tunnel, $10\frac{1}{4}$ miles of cut-and-cover work, and $3\frac{3}{4}$ miles of pipe siphons. The built and tunneled parts of the aqueduct (Fig. 165) are 8 ft. wide by 8 ft. high with an arched roof, and have a uniform grade of 10 ins. per mile. The siphon pipes are laid across three wide valleys, and have a mean fall of 1 in 1000. Two 48-in. cast iron pipes were placed originally in each siphon, and provision was made for a third line of somewhat smaller pipe, which was to be laid when needed.

There are about 25 important iron and stone bridges, some of them of considerable magnitude, on the line of the aqueduct. Minor ravines are crossed by bridges consisting of wrought iron tubes and cast iron troughs, covered with planking, but these bridges have not proved to be satisfactory. The service reservoir, which was formed by constructing two earthen dams, has a storage capacity of about 500,000,000 Imp. gals. of water. A basin having 4 cast iron gauge-plates, each 10 ft. long, was built at the upper end of the reservoir, and is used for measuring the delivery of the aqueduct, which has a maximum capacity of 40,000,000 Imp. gals. per day. The outlet of the reservoir is controlled by a stand-pipe having valves at different levels, and the water drawn from the

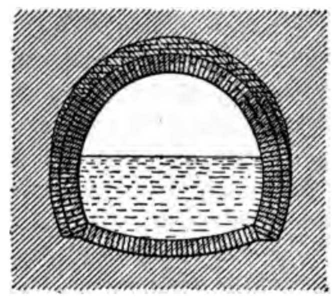
* Proc. Inst. M. E., 1864, p. 123.

reservoir has to pass through a straining well at the reservoir, before it is conveyed to the city. The Loch Katrine aqueduct and reservoirs were built according to the plans, and under the direction, of J. F. Bateman, M. Inst. C. E.

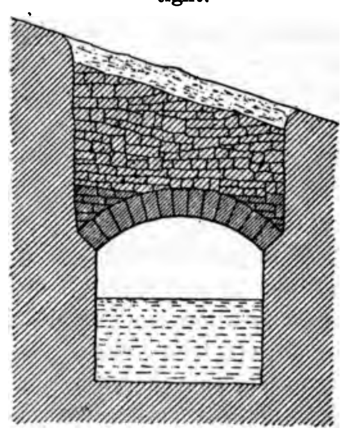
Glasgow obtained an additional supply from Loch Katrine by raising the loch 5 ft., which increased its available storage capacity to 9,849,000,000 Imp. gals., and by constructing a new aqueduct having a capacity of 70,000,000 Imp. gals. per day, from the loch to the city. This increase in supply of water was only obtained gradually. The construction of the works were begun in 1885, but was spread over a considerable period of time. The portions of the old aqueduct which offered the greatest obstruction to the flow of water were duplicated first, and the increase in the storage, and the laying of the pipes, were done from time to time, as the demand for water required. The new conduit was located almost parallel with the old one, but, by resorting largely to tunneling, the length of the new aqueduct was reduced to $23\frac{1}{2}$ miles, a saving of $2\frac{1}{4}$ miles in distance. The two aqueducts are connected at several points, in order to make it possible to isolate any section of either aqueduct for inspection or repairs. As the minimum available fall from the loch to the terminal reservoir is 38.58 ft., a fall of 1 in 5500 was obtained for the tunnels of the new aqueduct, and a fall of 1 in 960 for the siphon pipes. About 50 per cent of the tunnels are lined with concrete, and have a cross-section 9 ft. wide at the bottom, and 10 ft. wide below the springing of the arch. The invert has a sine of 6 ins. The unlined tunnels are 11 ft. wide at the bottom, 12 ft. wide at a height of 6 ft.



Tunnel in Rock.



Tunnel in Material not Water-tight.



Open Cutting in Rock.

FIG. 165. — Glasgow Aqueduct. (Built 1856–1860.)

above the bottom, and 9 ft. high, the roof being formed by a segment of a circle.

There are only five short aqueduct bridges on the line of the new conduit. They are all built of masonry. Two valleys, having respectively widths of 2.425 and 0.675 miles, are crossed by means of siphons, each consisting of two lines of 40-in. cast iron pipe. Provision was made for laying two additional lines of similar pipes, when required. The thickness of the walls of the pipes varies from $1\frac{1}{8}$ to $1\frac{5}{8}$ ins., according to the pressure to which they are subjected. In the wider valley, the pipes are carried across the railway, a public road, and a river on steel plate-girders, supported by masonry abutments. A new service reservoir, adjoining the older storage basin, was constructed at the same height above the sea. The new works were designed, and carried out, by James N. Gale, M. Inst. C. E.

256. Liverpool, England, obtained, in 1881 to 1890, a new water supply by building a masonry dam across the Vyrnwy River, in a remote part of North Wales, to form an impounding reservoir, and by constructing an aqueduct from this reservoir to the city.*

257. *Lake Vyrnwy,* as the storage reservoir is called, covers 1121 acres and stores, when full, about 13,000,000,000 Imp. gals. of water. The lake is $4\frac{3}{4}$ miles long, and has an average width of half a mile. The crest of the overflow is 826 ft. above the sea-level, and the greatest depth of water in the reservoir is 84 ft.

The lake is formed by a masonry dam, 1165 ft. long on top, having a maximum height of 161 ft. from the lowest part of the foundation to the top of the parapet of the carriage way. The greatest thickness of the base is 120 ft., and the width of the roadway on the top of the dam is 19 ft. 10 ins. The whole dam forms a spillway, with the exception of the piers which support the roadway. Elliptical arches of 24 ft. span are built between these piers. The overflow level is 10 ft. below the surface of the roadway.

During the construction of the dam, two circular outlet tunnels, 15 ft. in diameter and about 70 ft. long, were left in the masonry of the dam for the temporary diversion of the river. On the completion of the dam, the tunnels were closed by brick bulkheads, 18 ft. thick. A 39-in. blow-off pipe is placed in one of these tunnels, and a 30-in. and an 18-in. blow-off pipe are laid in the other tunnel. These pipes pass through the brick bulkhead and are controlled by suitable gate-valves.

258. *The Vyrnwy Tower* (Fig. 166). — The outlet from the lake is controlled in a circular tower, provided with the necessary inlet- and

* The Vyrnwy Works are fully described in a paper by George F. Deacon in Proc. Inst. C. E., 1895, CXXVI, p. 24.

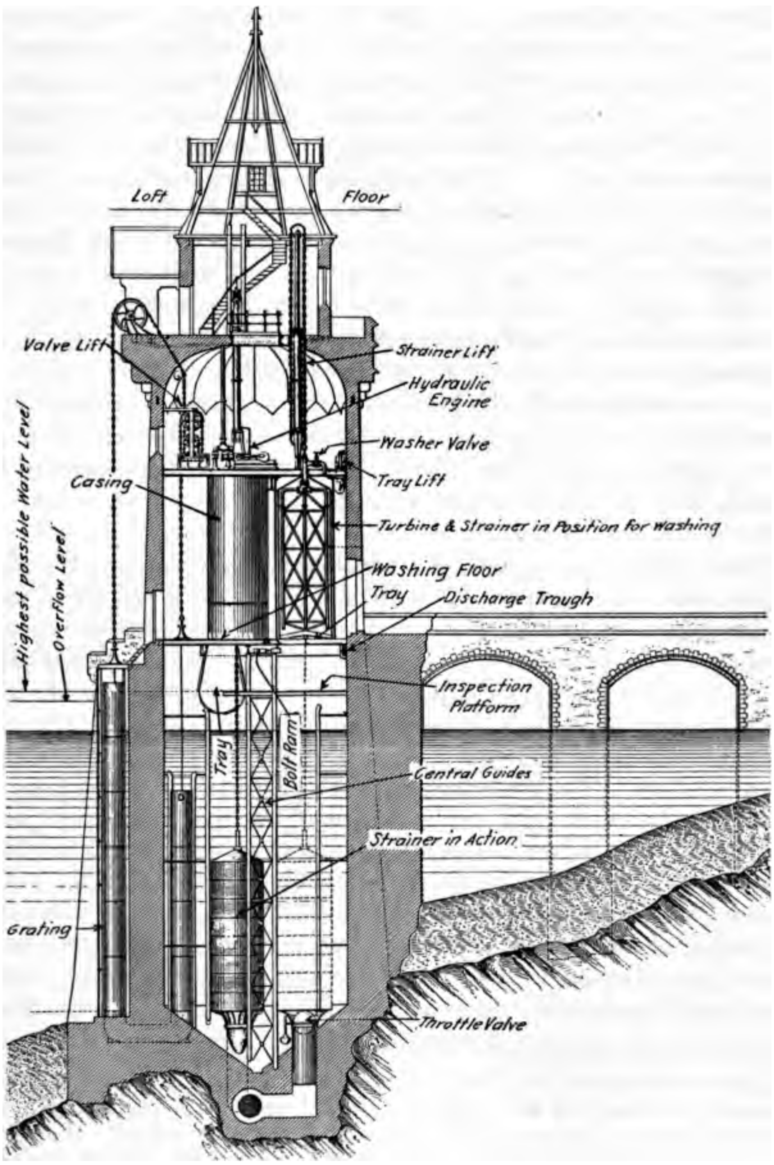


FIG. 166. — The Vyrnwy Tower.

outlet-gates, and with screens. In order to be able to draw deep lake water at a point where it would not be affected by floods, the tower was built off a promontory, about 2200 ft. northwest of the inlet of the aqueduct. It has a height of about 170 ft. above the base, and rises 110 ft. above the high-water level. The tower and the bridge that connects it with the shore are built of concrete. There are two inlet-valves, which admit the water into the tower. They are put on the outside of the tower between two side walls, and a grating is placed, from side wall to side wall, to keep coarse matter from getting into the tower. Each inlet-valve consists of six 9-ft. lengths of steel pipe, 36 ins. in diameter, with gun-metal faced ends. These pipes form a stand-pipe, extending from the lowest available water level to the highest flood level. By suitable attachments and a hydraulic hoisting gear, the stand-pipe can be raised so as to open any of its five joints to admit the water, the other joints remaining closed. At its bottom the stand-pipe is connected by a horizontal piece of pipe with a similar stand-pipe on the inside of the tower, having, however, less height. The two inside stand-pipes can be opened, like the outside pipes, so as to discharge the water into the tower at any desired joint.

Three cylindrical strainers, of a novel design, are placed in the water in the tower. The lower end of each strainer is provided with an India rubber ring, to make a tight joint, and rests upon a bell-mouthed casting, forming the upper end of a vertical cast iron pipe, 46 ins. in diameter, which is provided with an ordinary gate-valve, that controls the flow from the strainer. Each strainer consists of a cylindrical frame of wrought iron, 25 ft. high and 9 ft. in diameter, over which fine copper gauze is placed. The mesh first used was 14,400 to the square inch, but it was found to be too fine and too costly. Different meshes were tried and the effect on the removal of the suspended matters noted. Eventually a mesh of 60 to the lineal inch was adopted. The strainer is held in place by three equidistant brackets, which serve as guides when the strainer is hoisted to the floor of the tower, to be cleaned by revolving water jets. During the washing process, a tray, lifted into position immediately under the strainer, catches all matter washed off the gauze. A pipe conveys the washed material to a safe place outside the tower.*

* When the strainers were first put into service, a galvanic action took place between the copper gauze and the iron to which it was attached, which rapidly corroded the copper. This action was caused by a certain amount of acidity in the water, or in the matter in contact with the strainer. In ordinary cases, copper is electro-negative to iron, but in this case this condition was reversed, probably on account of the thinness of the copper wire, and the great extent of its surface.

If the copper gauze had been fastened to wooden frames, and thus kept from contact with the iron, this difficulty would not have occurred. Strips of zinc, hung

The three 46-in. pipes which receive the water from the strainers are connected with a concrete culvert that conveys the water to the inlet of the aqueduct. Here there is a shaft with a valve for controlling the supply from the tower. In the same shaft, there is a pipe through which a supply can be drawn, at a somewhat lower level, independently of the tower.

259. *The Vyrnwy aqueduct* has a length of about 68 miles from Lake Vyrnwy to the Prescot service-reservoirs. With the exception of three tunnels, driven through rock, the aqueduct consists of siphons of cast iron pipes, usually 42 ins. in diameter. For the full capacity of the aqueduct, which was fixed as 40,000,000 Imp. gals. per day, three lines of 42-in. pipe are required. Only one line was laid as a first instalment, the other two being placed, as required. Four open balancing reservoirs or tanks are built at convenient points on the line of the aqueduct. They divide the aqueduct into the six following divisions:

TABLE XLII. — VYRNWY AQUEDUCT

Divisions	Miles
1. Lake Vyrnwy to Parc Uchaf Reservoir	10
2. Parc Uchaf Reservoir to Oswestry Reservoir	8
3. Oswestry Reservoir to Malpas Tank	18
4. Malpas Tank to Cotebrook Reservoir	12
5. Cotebrook Reservoir to Norton Tank	11
6. Norton Tank to Prescot Reservoir	9
Total	68

NOTE. — An automatic Armstrong gate-valve is placed in the outlet pipes from each of the reservoirs or tanks. For a description of these gate-valves see p. 183.

260. *Tunnels.* — The aqueduct begins at the lake with a circular tunnel, $2\frac{1}{4}$ miles long, driven through the ridge that separates the lake from the valley of the Tanat. The inlet of the tunnel is connected with the straining tower by means of a brick culvert, and a line of cast iron pipes is laid from the tunnel for a certain distance on the bottom of the lake, to make it possible to draw water independently of the

within the screens and in contact with the copper, did much to save the gauze by creating an electro-positive, which could be replaced. By using heavier wire, the life of the copper wire was much increased, but this caused a reduction of the meshes from 14,400 to 10,000 per square inch.

The difficulty was finally successfully overcome by Joseph Parry, M. Inst. C. E., who later became chief engineer of the Liverpool Water Works, by placing sheet copper between the iron and the copper gauze, in contact with both. The greater mass in relation to its surface of this added copper, if it did not create a balance, caused, no doubt, decomposition of the iron, rather than of the copper.

straining tower. For 1890 ft. of its length, the tunnel is lined with masonry, the inner diameter being 7 ft. For the remaining distance, the rock is excavated so as to clear a 7-ft. cylinder by 2 ft. The invert has a fall of 2.04 ft. per mile. There are two other tunnels on the line of the aqueduct, each a little less than a mile in length, and having a grade of 2 ft. per mile. For convenience we shall number the tunnels in the following descriptions, beginning at the lake.

261. *Siphons.* — There are seven siphons of cast iron pipes, each of which consists for the full delivery of the aqueduct of three lines of cast iron pipes, usually 42 ins. in diameter. For the first instalment only one line of pipes was laid. The lengths, grades, etc., of the different siphons are given in the following table.

TABLE XLIII. — SIPHONS OF VYRNWY AQUEDUCT

Location	Maximum pressure, ft.	Length, miles	Grades per mile
Tunnel No. 1 to Parc Uchaf Reservoir	317	7.00	4.5
Parc Uchaf Reservoir to Tunnel No. 2	220	6.25	4.5
Tunnel No. 2 to Tunnel No. 3	....	0.10	....
Oswestry Reservoir * to Malpas Reservoir	480	17.63	6.87
Malpas Reservoir to Cotebrook Reservoir	308	11.63	4.6
Cotebrook Reservoir to Norton Tank	390	11.00	4.8
Norton Tank to Prescot Reservoir	363	9.25	6.0

* The diameter of the pipes was reduced in this siphon to 39 ins., on account of the head to which it is subjected. The fall per mile was increased to get the required delivery.

With few exceptions, the siphons are made of cast iron hub-and-spigot pipe, 42 ins. in diameter and 12 ft. long. The thickness of metal of the pipes varies from 1 to $2\frac{1}{4}$ ins., according to the pressure that has to be resisted. Blow-off valves for emptying the siphons are provided at all depressions, and automatic valves for releasing any air that may collect at summits of the pipe-line, are placed at all high points, where the pipes dip to a lower level. Improved self-acting gate-valves (p. 183) are placed in the pipe-lines at intervals of less than five miles. In addition to these valves, ordinary stop-gates are put at frequent intervals in the pipe-lines, to permit the control of the flow through the aqueduct.

At a number of places the siphons had to cross streams, railroads, and canals. In ordinary cases the pipes were laid in the beds of the streams or canals, and in subways under railroads. In one instance, where the railroad was in excavation, the aqueduct was formed of riveted wrought iron pipes, placed $16\frac{1}{2}$ ft. above the rails, and held by suitable supports.

The most important crossings on the line of the aqueduct are those of the River Weaver Navigation, the Manchester Ship Canal, and the River Mersey. The River Weaver Navigation, which is a waterway 100 ft. wide and 15 ft. deep, was crossed by means of three lines of straight steel pipes, each 107 ft. long, which were connected together laterally. The pipes were floated, with their ends temporarily closed, into position and sunk into the trench, which had been previously dredged to a depth of 23 ft. below the water level. After being placed, the tubes were covered with 18 ins. of concrete.

The Manchester Ship Canal was crossed during its construction by *cut-and-cover work*. A circular culvert of brickwork, with terminal shafts, 305 ft. apart, was built under the bed of the canal. It provides room for three lines of steel pipe, 36 ins. in diameter, one line of which was laid as the first instalment. The culvert inclines towards the Norton tank, and, in the shaft, nearer this tank, the gate-valves of the pipe-lines are placed. A pumping plant, worked by the pressure in the mains, is installed here for keeping the culvert dry.

Immediately after passing the ship canal, the River Mersey had to be crossed. This involved the most difficult piece of work on the line of the aqueduct. It necessitated driving a tunnel under the Mersey River for the siphon pipes. This was the first tunnel excavated through loose material under a river, by means of a shield. The original plans contemplated laying the pipes on the bed of the river, as has often been done, but Parliamentary exigencies obliged the City of Liverpool to construct a tunnel. It has a diameter of 9 ft., within the flanges of the cast iron plates with which it is lined. The external diameter of the tunnel is 10 ft. One line of 32-in. steel pipes was laid in the tunnel for the first instalment. Provision was made for a second line of pipe. The greatest head on the siphon is 363 ft. The tunnel is very dry.

262. *Balancing Reservoirs.* — The capacities of the different balancing reservoirs, as finally completed, are as follows: Uchaf, 6,000,000 Imp. gals.; Oswestry, 46,000,000 Imp. gals.; Malpas, 6,500,000 Imp. gals.; Cotebrook, 6,000,000 Imp. gals.; Norton tower, 651,000 Imp. gals.; Prescot, 88,000,000 Imp. gals. With the exception of the Norton tank, the reservoirs are ordinary storage basins, built by excavation and embankment.

Norton tower (Fig. 167) was built of red sandstone on a high hill, about midway between the Cotebrook and terminal reservoirs, as no suitable ground could be found for a reservoir. The tower has a diameter of 95 ft. at the base and a height of 113 ft. to the top of an iron water tank, which is supported on the circular cornice of the tower, without any girders or columns. The tank is 80 ft. in diameter and has a depth of 31 ft. at the center. The upper 10½ ft. of the tank is formed of a cast iron cylinder which is supported on steel rollers, placed on a cast iron bed-plate on the cornice of the tower. The lower part of the tank consists of a segment of a sphere, made entirely of steel plates, which is

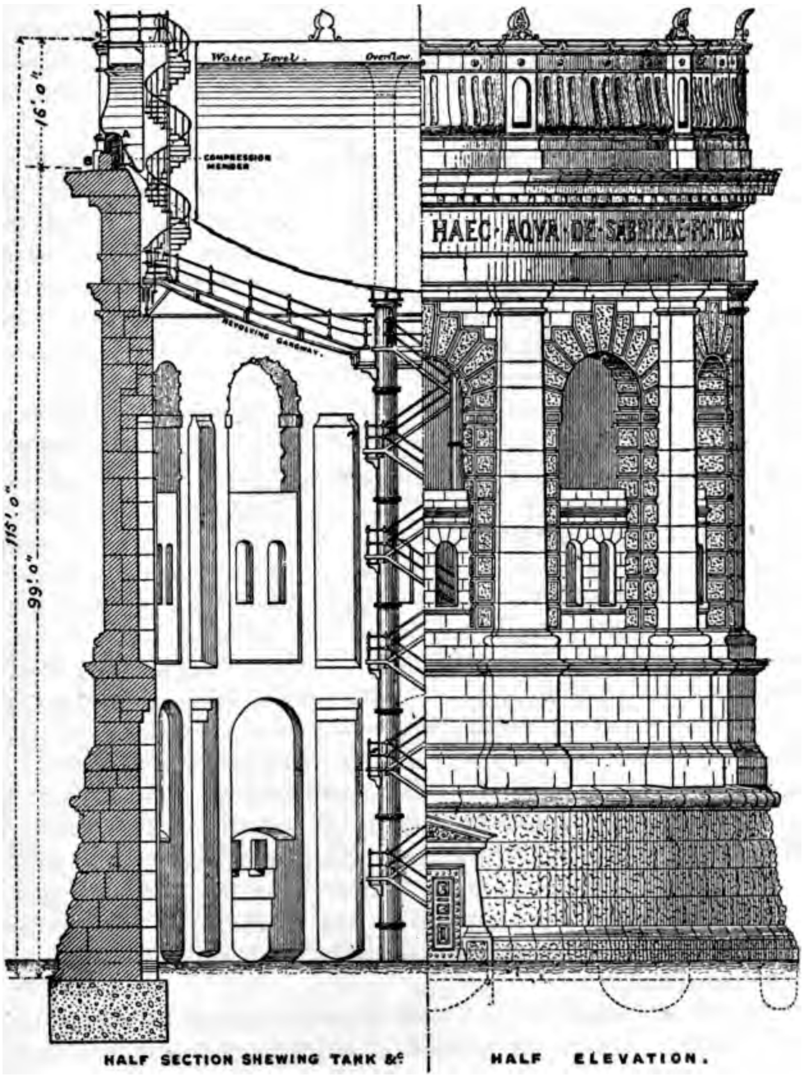


Fig. 167. — The Norton Tower.

connected with the cast iron cylinder. Stand-pipes from the three pipe-lines of the siphon, and, also, an overflow-pipe, are placed in the tower, and are connected by means of suitable packing-glands with the bottom of the tank. The tower is built of Roman Doric design of ashlar masonry throughout.

The two Prescot reservoirs, at which the aqueduct terminates, were built in 1855–57, in connection with the Rivington works, with the high water mark 275 above sea level. From these reservoirs the water is conveyed by gravitation, with one exception, to a number of covered service-reservoirs, built of brickwork or concrete on high ground, in different parts of the city. Filter-beds, of ordinary construction, are built about $\frac{3}{4}$ mile from the Oswestry reservoir, the fall in this distance to the filters being 112 ft.

263. *Engineers.* — The construction of the works was carried out according to the plans, and under the supervision, of Thomas Hawksley and George F. Deacon, members Inst. C. E., until 1885, when Mr. Hawksley retired and Mr. Deacon became the sole engineer in charge of the works to their completion in 1890.

264. The Thirlmere aqueduct of Manchester, England,* was constructed in 1886–1895, to obtain a water supply from Lake Thirlmere in the English Lake District, about 96 miles from Manchester. By building a masonry dam, 880 ft. long on top and 114 ft. high above the lowest part of the foundation, the surface of the lake was raised 50 ft. and its storage capacity increased to about 8,131,000,000 Imp. gals.

The aqueduct, which was designed for a maximum discharge of 50,000,000 Imp. gals. per day, has a length of 95$\frac{7}{8}$ miles from the lake to the city. It consists of 14$\frac{1}{8}$ miles of tunnels, 36$\frac{3}{4}$ miles of cut-and-cover work, and 45 miles of siphons of cast iron pipes. Each siphon is to be eventually composed of several lines of pipes. Only one line of pipe was laid as the first instalment. Each of the first two siphons is to have three lines of 48-in. pipes, for the full capacity of the aqueduct, but the other siphons are to consist of five lines of 40-in. pipes, except for the last 13 miles of the aqueduct to the Prestwich reservoir, where the diameter of the pipes is reduced to 36 ins., as a greater fall is available. Each siphon is connected at each end to the masonry conduit by a small masonry chamber. Armstrong automatic gates are placed in these chambers for each pipe-line.

The aqueduct begins at the lake with a tunnel. At a distance of 300 ft. from the inlet, a circular straining well is constructed. It was sunk 65 ft. through solid rock, and was lined with concrete. The well is 37$\frac{1}{2}$ ft. in diameter at the bottom and 40$\frac{1}{2}$ ft. at the top, the reduction in diameter being made in two off-sets. On the lake side of the well, a rectangular valve-shaft was sunk to the same depth as the well and

* "The Thirlmere Works for the Water Supply of Manchester," by George Henry Hill in Proc. Inst. C. E., 1895, CXXVI, p. 2.

was, also, lined with concrete. The tunnel ends at the valve-shaft and the water is conveyed from this shaft to the straining well by a 42-in. cast iron pipe, controlled by two gate-valves. The water is delivered in the straining well on the outer side of an octagonal screen of strainers, 22 ft. in diameter and 16 ft. high. The aqueduct (Fig. 168) is built from the straining well in tunnel for about 3 miles. The cut-and-cover portion of the aqueduct is carried across streams on masonry bridges, at which waste weirs and blow-off gates are provided. The grade of the aqueduct, except at the siphons, is 20 ins. per mile. Manholes with ventilators are constructed at intervals of about $\frac{1}{4}$ mile.

The Thirlmere Water Works were designed and constructed under the direction of George Henry Hill, chief engineer.

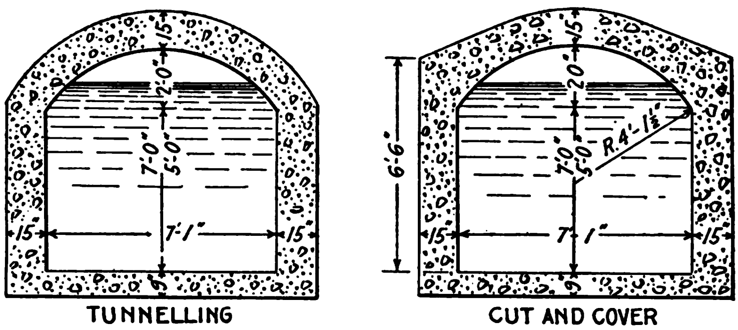


Fig. 168. — The Thirlmere Aqueduct.

265. Birmingham, England,* secured, in 1894–1906, a supply of water from the Elan and Claerwen Rivers, tributaries of the Wyem, in central Wales. The city purchased practically the whole water-shed of these streams above their confluence, which is inaccessible, almost uninhabited, and forms an ideal collecting ground for a supply of pure, soft water.

The water from this drainage area is collected in several storage reservoirs. The outlet from the lowest reservoir is controlled by a valve-tower, constructed somewhat similar to the Vyrnwy tower. From the valve-tower the water is conveyed by a circular rock tunnel, 8 ft. in diameter and about 7000 ft. long, to filter-beds, where it is discharged into an open channel, passing along the side of the filter-beds. After being filtered, the water flows through a measuring chamber and then into the main aqueduct, which conveys it to a receiving reservoir

* "The Engineer," London, Sept. 9, 1898, p. 243; Oct. 12, 1900, p. 362; July 22, 1904, p. 77.

at Frankley, near the city. The aqueduct, which is $73\frac{1}{4}$ miles long, is built for $23\frac{3}{4}$ miles as cut-and-cover work, for 12 miles in tunnel, and for $37\frac{1}{2}$ miles as siphons, each of which is to consist ultimately of 6 parallel lines of cast iron pipes, 44 ins. in diameter. Only two lines of pipes were laid originally in each siphon. The longest tunnel has a length of $4\frac{1}{4}$ miles and a grade of 1 in 3016. The longest siphon is built across the valleys of the Severn and Stour. It is about 17 miles long, has a dip of 547 ft., and a grade of 1 in 1600. For the cut-and-cover work the grade is 1 in 4000. The average fall for the whole aqueduct is 1 in 2276.6. The conduit has a maximum delivery of 75,000,000 Imp. gals. per day.

266. *Engineers.* — The works described above were planned and carried out by William Gray, the engineer in charge of the water works

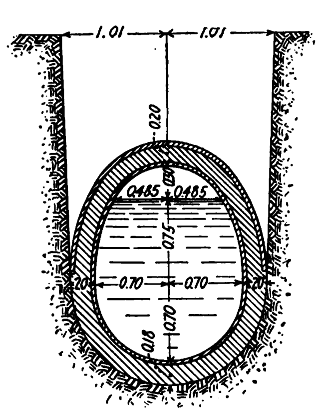


FIG. 169. — The Dhuys Aqueduct. (Dimensions in Meters.)

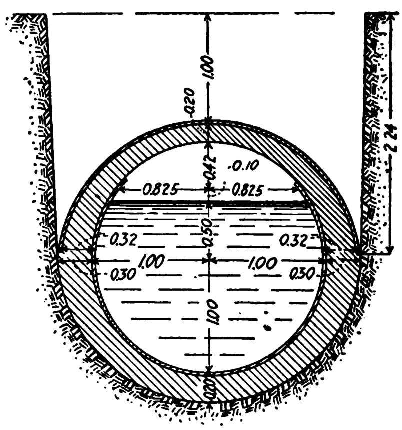


FIG. 170. — The Vanne Aqueduct. (Dimensions in Meters.)

of the city, and James Mansergh, M. Inst. C. E., acting as consulting engineer.

267. Paris, France, has a double water system, spring water being distributed for domestic consumption, while river water is used for public purposes. The domestic supply is conveyed by four aqueducts, described briefly below.

268. *The Dhuys aqueduct** (Fig. 169) was built in 1863–66 to obtain a water supply from three springs, about 80 miles east of the city, the lowest of which is 394 ft. above the level of the sea. Some additional springs were diverted later into the aqueduct. The conduit is built, according to different types of construction, as follows:

* "Les Eaux Nouvelles," by M. Belgrand, p. 100; Paris, 1882.

	Miles
Cut-and-cover conduit	63.34
Tunnels	7.58
Siphons	10.56
Total	81.48

The aqueduct has 22 tunnels; 21 siphons, each consisting of a single line of cast iron pipe, 3.28 ft. in diameter; 21 bridges; 216 manholes, built of dry masonry, 1640 ft. apart; and 46 gate-houses, to control the flow of the water. It begins at a large inlet gate-house having a well into which the water from the different springs is led in cast iron pipes. For about 2.4 miles from this gate-house, the aqueduct is built as a double conduit to the up-stream gate-house of the first siphon. From the down-stream gate-house of this siphon the aqueduct consists, for about 7500 ft., of a masonry conduit having an egg-shaped cross-section, 3.94 ft. wide by 5.38 ft. high. At the end of this section water from additional springs is led into the aqueduct and its cross-section is, therefore, enlarged to a width of 4.59 ft. and a height of 5.77 ft.

The grade of the aqueduct is uniformly 1 in 10,000, except on the siphons where it is increased to 5.5 in 10,000. The water obtained by this aqueduct — about 5,000,000 to 6,000,000 gals. per day — is discharged into the upper basin of the 2-story service-reservoir of Menilmontant, the lower basin being used for storing water from the river Marne, for public purposes (p. 353).

The aqueduct was built according to the plans, and under the direction of, M. Belgrand, for many years chief engineer of the Waterworks of Paris.

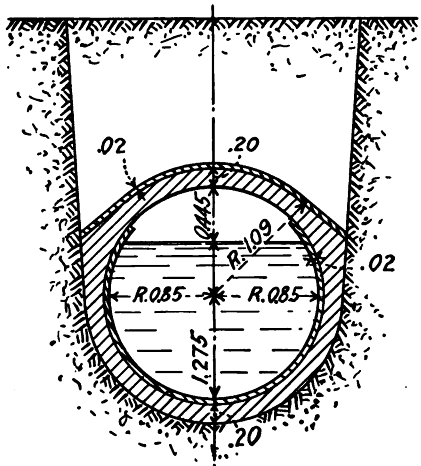
269. *The Vamne aqueduct* * (Fig. 170) was built in 1867–77, to obtain a supply of water from springs in the water-shed of the river Vamne, located about 85 miles southeast from Paris. The city bought most of the available springs in this water-shed, and laid pipes, or built egg-shaped masonry feeder conduits, 2.62 by 3.51 ft. or 4.59 by 5.74 ft., to deliver the water into the Vamne aqueduct. Some of the springs are 260–460 ft. above tide, but others are at lower levels, and their water has to be pumped into the conduit.

The water of the springs is delivered by an auxiliary conduit, called the *collector*, to the main aqueduct. The collector, which is 12.66 miles long, is built in the same manner as the main aqueduct with tunnels, arcades and siphons, but with a steeper grade, namely, 2 in 10,000. Its diameter is 5.58 ft. for the upper part and 5.90 ft. for the lower part. The walls of the collector are uniformly 8 ins. thick, and are plastered, on the inside, with cement mortar, $\frac{3}{4}$ in. thick.

* "Les Eaux Nouvelles," by M. Belgrand, p. 146; Paris, 1882.

The main aqueduct consists of the following parts:

	Miles
Cut-and-cover conduit	35.25
Tunnels	23.68
Arcades	15.42
Siphons	10.38
Total	84.73



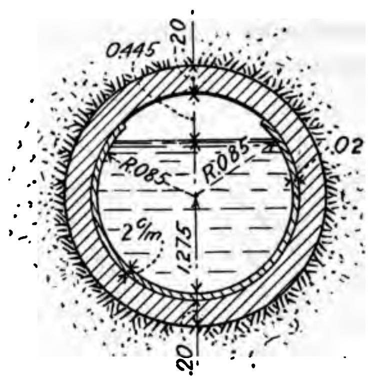


FIG. 171. — Avre Aqueduct in Trench. FIG. 171*a*. — Avre Aqueduct in Tunnel.
(Dimensions in Meters.)

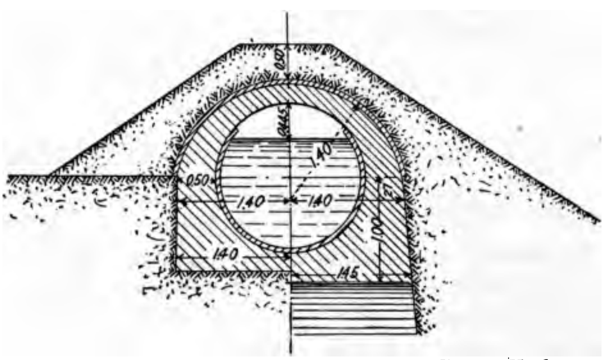


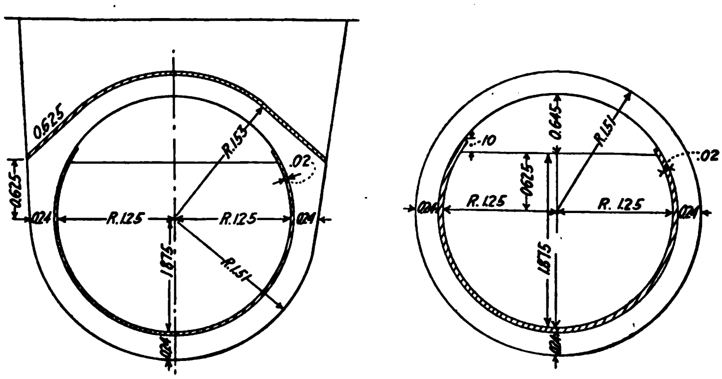
FIG. 172. — Avre Aqueduct in Cut-and-Cover Work.
(Dimensions in Meters.)

The masonry conduit has a circular cross-section, the diameter for the upper part being 6.56 ft. and, for the lower part, 6.89 ft., exclusive of a cement coating, $\frac{3}{4}$ in. thick, plastered on the inside of the conduit. The grade of the conduit is 1.3 in 10,000 as far as Orige, and 1 in 10,000 for the remaining distance to Paris.

Many of the siphons are supported on arcades having a single or a double tier of arches. The longest siphon, which has a length of 2.32 miles, is carried by 162 masonry arches. The manholes and gate-houses are similar to those of the Dhuys aqueduct. The water is delivered into the Montsouris reservoir in Paris at an elevation of 262.4 ft. above tide.

The aqueduct was built according to the plans, and under the direction of, M. Belgrand, chief engineer of the Waterworks of Paris.

270. *The Avre aqueduct** (Figs. 171–173) was built in 1891–1893 to bring to the city water from 6 springs, located about 60 miles west of Paris. The quantity of water which the city is authorized to draw from these springs is limited to 338 gals. per second. The waters of the springs are collected by two auxiliary conduits, about 4600 and 5600 ft.



In Cut. In Trench.

FIG. 173. — Loring and Lunain Aqueduct.
(Dimensions in Meters.)

long respectively, which converge at the inlet gate-house of the aqueduct.

The main aqueduct is composed of:

	Miles
Cut-and-cover conduits	40.51
Tunnels	15.83
Siphons	4.65
Arcades	2.42
Total	63.41

The aqueduct has a circular cross-section, the thickness of the masonry being about 9 ins. For the first 11.8 miles, the diameter of the conduit is 5.58 ft., and the grade is 4 in 10,000. For the remaining distance, the diameter is 5.90 ft., and the grade is 3 in 10,000, the total fall being 129.17 ft.

* "Annales des Ponts et Chaussées," 2. Trimestre, 1902, p. 131.

Each siphon consists of two lines of cast iron pipes, 3.28 ft. in diameter, laid 16.4 ft. apart, and 3.28 ft. below the surface of the ground. The two lines of pipe deliver about 40,000,000 gals. per day. The aqueduct has the same system of gate-houses and manholes that was adopted for the earlier conduits.

The works were built by M. Bienvenue, chief engineer, under the direction of M. Humbolt, general inspector.

271. *Loring and Lunain aqueduct* * (Figs. 173 and 174), constructed in 1898–1902, obtains its supply from springs in the water-sheds of the Loring and Lunain Rivers. These springs are all too low for a gravity supply. The waters of the springs are conveyed by small conduits to a

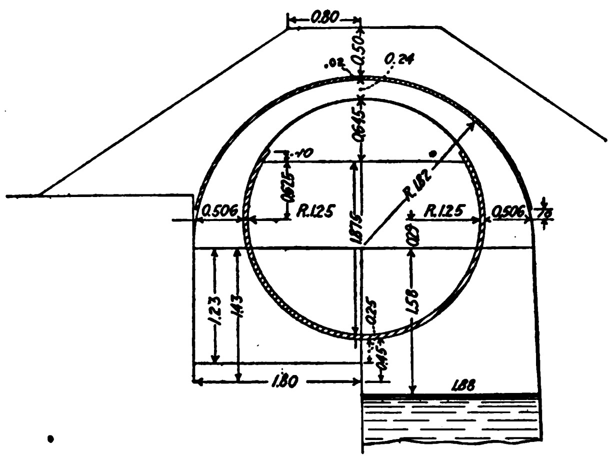


FIG. 174. — Loring and Lunain Aqueduct in Cut-and-Cover Work. (Dimensions in Meters.)

pumping station at Sorques, at the edge of the forest of Fontainebleau, and pumped to an elevation of 134.48 ft. above tide.

The main aqueduct begins at the pumping station, and is built for the first 46.6 miles practically parallel with the Vamne aqueduct, the two conduits being generally about 10 meters (32.8 ft.) apart. The new aqueduct is at first on the west side of the Vanne conduit, but crosses it twice, nearer the city, and, before reaching the city, it diverges from the older conduit.

The aqueduct is a circular conduit having an inner diameter of 8.2 ft. in the clear. The masonry of the conduit is $9\frac{7}{16}$ ins. thick, and is plastered on the inside with $\frac{3}{4}$ in. coating of cement mortar. When the aqueduct is utilized to its full capacity, it will deliver about 47,500,000 gals. per day.

* "Annales des Ponts et Chaussées," 3. Trimestre, 1905, p. 5.

272. Aqueducts of Vienna, Austria. — The first of these conduits — called the Emperor Francis Joseph High Springs aqueduct (Fig. 175) — was constructed in 1868–73 to convey the water of two large springs in the foot-hills of the Styrian Alps to Vienna. The springs are, respectively, 1196 ft. and 913 ft. above sea-level. A covered masonry chamber was constructed at each spring, to collect its water and that of water veins nearby. Including the branch conduit to the lower spring, the aqueduct has a total length of 58.86 miles. It terminates at a receiving reservoir, built at the southern boundary of the city. The grades vary on different portions of the conduit from 1 in 310 to 1 in 2200. In all parts, the aqueduct is made large enough to be passable, for inspection and repairs.

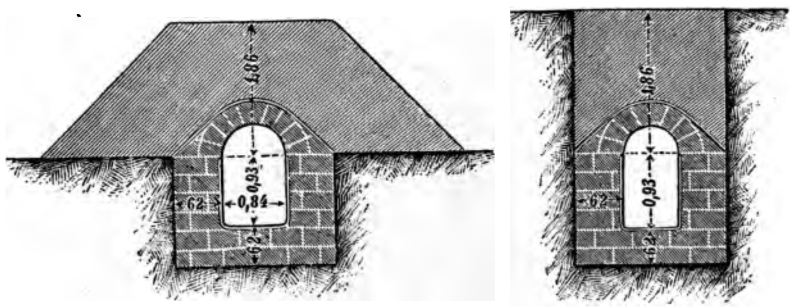


FIG. 175. — Aqueduct of Vienna, built 1868–73.
(Dimensions in Meters.)

With the exception of 9 per cent of its length in tunnels and about 5 per cent in aqueduct bridges at valley crossing, the aqueduct is built as cut-and-cover work. In connection with the aqueduct, 2.4 miles of retaining walls, 22 culverts, and 10 bridges had to be built. From the receiving reservoirs the water is conveyed by 2 lines of 36-in. cast iron pipes to 3 covered service-reservoirs, built of masonry and turfed over.

The plans of the aqueduct and the distribution system in the city were prepared by Karl Junker and Karl Gabriel. The former had charge of the construction of the aqueduct, as chief engineer; and the latter was chief engineer of the distributing system to 1865, when he resigned and was succeeded by Otto Wertheim.

Karl Mihatsch, who, as chief engineer of the Municipal Bureau of Buildings, had a great deal to do with the design of the works, wrote a very comprehensive description of the aqueduct and distributing system.*

* "Der Bau der Wiener Kaiser Franz Joseph Hoch Quellen Wasserleitung," von Karl Mihatsch, Oberingenieur des Stadtbauamtes, etc., Wien, 1881.

PLATE V.

"High Bridge" and High Service Tower, at Harlem River, New York.

The supply of water furnished by the two springs mentioned above soon became insufficient for the rapidly growing population of Vienna. An additional supply was obtained by extending the aqueduct so as to obtain water from other springs, which were connected by pipe-lines or short tunnels with the aqueduct, and by pumping ground-water into the conduit. Even with these additions the water supply was not large enough for the city, and the construction of a second aqueduct was decided upon. The new conduit, which draws its supply from an additional water-shed, was built in 1901–1910.

273. *The second Emperor Franz Joseph High Springs aqueduct* * was built in 1901–10. As originally constructed, it is composed of the following parts:

	Miles
Masonry conduit	51.69
Tunnels	43.92
Siphons	12.80
Aqueduct bridges	2.73
Pipes, etc	2.23
Total	113.37

The conduit terminates in a masonry chamber, on a hill in the city, which is connected with a new service-reservoir, and, also, with the older basins.

The plans for the second aqueduct were prepared under the direction of Francis Berger, Director of the Bureau of Buildings of Vienna. He was assisted by engineers Sykora, Kinger, and Winterberg, who, later, had charge of the construction of the works.

274. The Old Croton Aqueduct.† — In 1837–42 the city of New York built an aqueduct, about 41 miles long, to obtain a supply of water from the Croton River, an affluent of the Hudson River. By building a masonry dam, about 50 ft. high, a reservoir storing 600,000,000 gals. was formed. The aqueduct — known now as the old Croton aqueduct — was built, from an inlet gate-house at the reservoir to a receiving basin of 180,000,000 gals. capacity, as a masonry conduit, with the exception of two siphons of pipes, one across the Harlem River on top of High Bridge (Plate V), and the other across Manhattan Valley. From this reservoir the water was conveyed by 3 lines of 36-in. cast iron pipes to a distributing reservoir, constructed on Murray Hill, then beyond the north boundary of the city.

* "Oesterreichische Wockenschrift für den öffentlichen Baudienst" für 1903 (Heft 42 S. 672 und Heft 29 S. 467).

† "Description of the New York Croton Aqueduct," by F. Schramke, New York, 1846.

The masonry conduit (Fig. 176) was generally built as cut-and-cover work. As a protection against frost, the conduit was covered with 3–4 ft. of earth. For an aggregate length of 6841 ft., it was built in short tunnels, driven through spurs of ridges. At the crossing of valleys the conduit was carried by walls of dry rubble, 11–15 ft. wide on top, and having on each side an earthen embankment, formed in layers, carefully rolled, and brought up simultaneously with the dry wall. This type of construction caused much trouble in later years, when the aqueduct was forced to convey more water than was originally intended. Leakage frequently occurred at the embankment and caused settling.

The grade of the conduit from the inlet to the south side of Manhattan Valley is uniformly $13\frac{1}{4}$ ins. per mile, except for a short distance

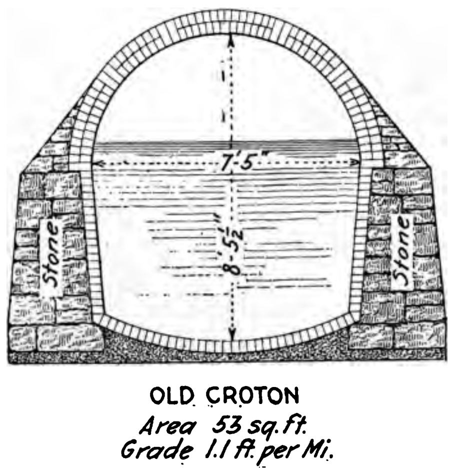


FIG. 176. — The Old Croton Aqueduct.
(Built 1837–42.)

at the inlet and at the two siphons, where additional heads, of 2 and 3 ft. respectively, were allowed. From the south side of this valley to the receiving reservoir, the grade was reduced to 9 ins. per mile. This part of the conduit was replaced in later years by cast iron mains.

Two lines of 36-in. cast iron pipes were laid originally across High Bridge in a masonry vault. Provision was made for two additional lines of pipes of the same diameter, but, instead of them, one riveted wrought iron pipe, $90\frac{1}{2}$ ins. in diameter, was placed in 1860–62 on the bridge between the two lines of cast iron pipes. Additional pipe-lines have, also, been laid across Manhattan Valley.

At six suitable places on the line of the aqueduct, gate-houses with waste-weirs and blow-off gates were constructed, and ventilator shafts were built at intervals of a mile. The surface drainage was passed under or over the conduit by 114 culverts, with spans of 18 ins. to 25 ft.

The conduit is carried across Sing Sing Kill on a masonry arch of 88 ft. span and 33 ft. rise, and across the Harlem River, on High Bridge.

By the construction of a high masonry dam across the Croton River, below the old dam, the high water mark of the old reservoir has been raised about 30 ft. The flow through the aqueduct described above is now controlled in a gate-house built at the new dam, or in a large gate-house, constructed on a bluff near the site of the old dam, to regulate the flow through the two Croton aqueducts.

The preliminary studies and surveys for the old aqueduct were made under the direction of Major D. B. Douglass, and the works were executed under the supervision of J. B. Jervis, chief engineer.

275. The New Croton aqueduct* was built in 1885 to 1891, to obtain an additional supply of water from the Croton River for the city of New York. To secure the required quantity of water, a number of storage reservoirs were built, in succession, on the affluents of the Croton River, and the capacity of the original reservoir on the Croton River was increased to about 34,000,000,000 gallons, by building a high masonry dam across the valley of the river, about 3 miles below the first dam. The aqueduct begins at a large inlet gate-house, built at the Croton reservoir.

276. *The inlet gate-house* (Figs. 177 to 177d) was built on the south side of the Croton River on the rocky hill-side, just below the old dam. This made it possible to supply the city with water through the new aqueduct, during the long delay that occurred before the new Croton dam was built. To provide the space required for the large gate-house, an area of about 90 by 100 ft. had to be excavated to a depth of 90 to 160 ft.

The gate-house has five inlets: The by-pass, 624 ft. long, connecting it with the reservoir above the old Croton dam; a surface, a middle and a bottom inlet, which are short circular conduits drawing water at different elevations at the gate-house; and a connection with the old Croton aqueduct through which water can be drawn into the gate-house from a small gate-house, built at the new Croton dam, three miles further down-stream. The inlets are 14 ft. in diameter, except the connection with the old Croton aqueduct, which has a diameter of 8.5 ft.

The aqueduct begins in the southwest corner of the gate-house, with its invert 60 ft. below the flow-line of the new reservoir. As the aqueduct

* Reports of the Aqueduct Commissioners, City of New York, 1887, 1895, and 1907; also, "The Water Supply of the City of New York, 1658–1895," by Edward Wegmann, C.E., New York, 1896.

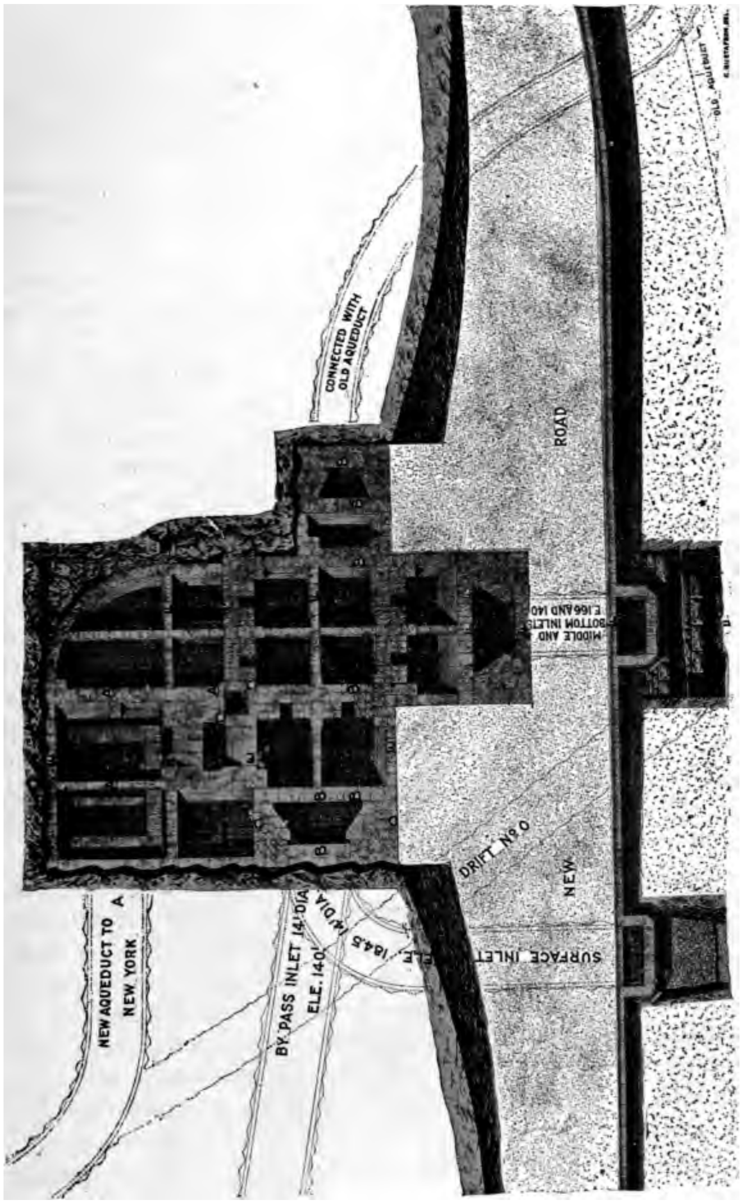
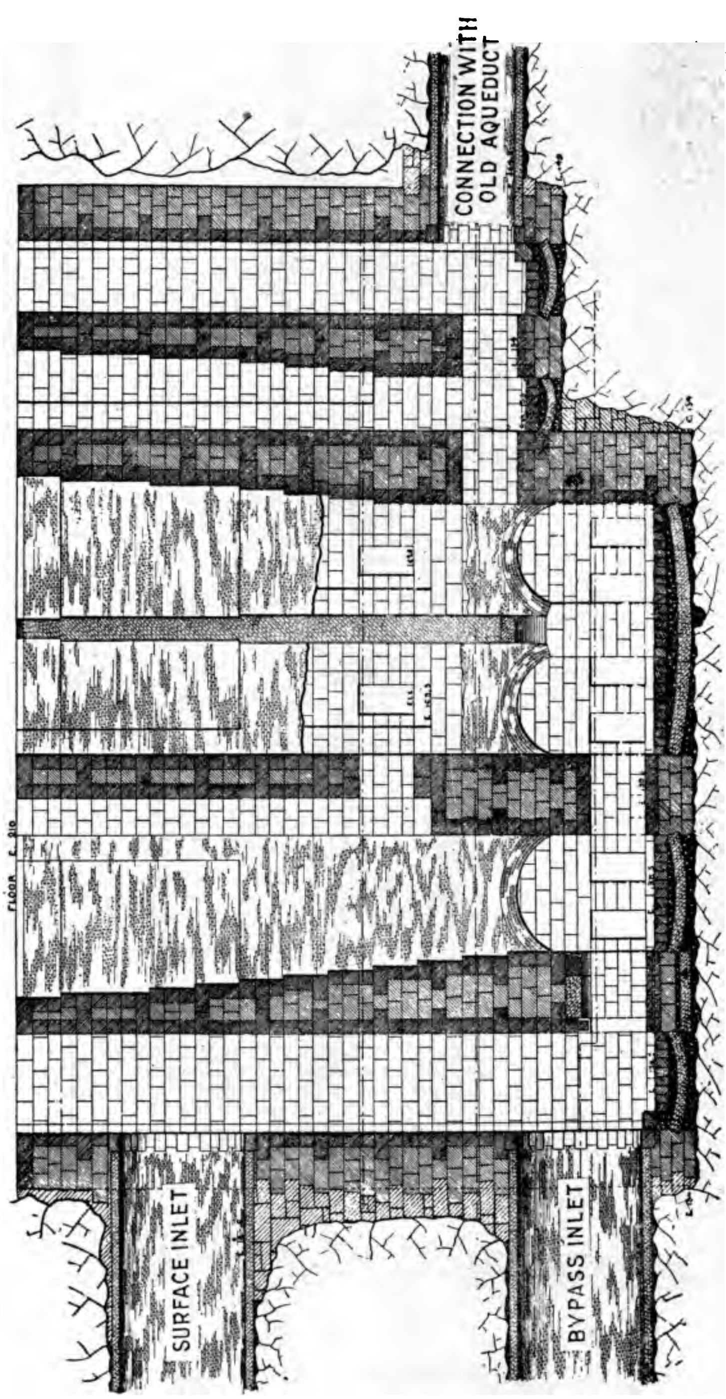


Fig. 177. — Plan of Inlet Gate-house of New Croton Aqueduct.



SECTION ON B.B.

FIG. 177*a*. — Inlet Gate-house of New Croton Aqueduct.

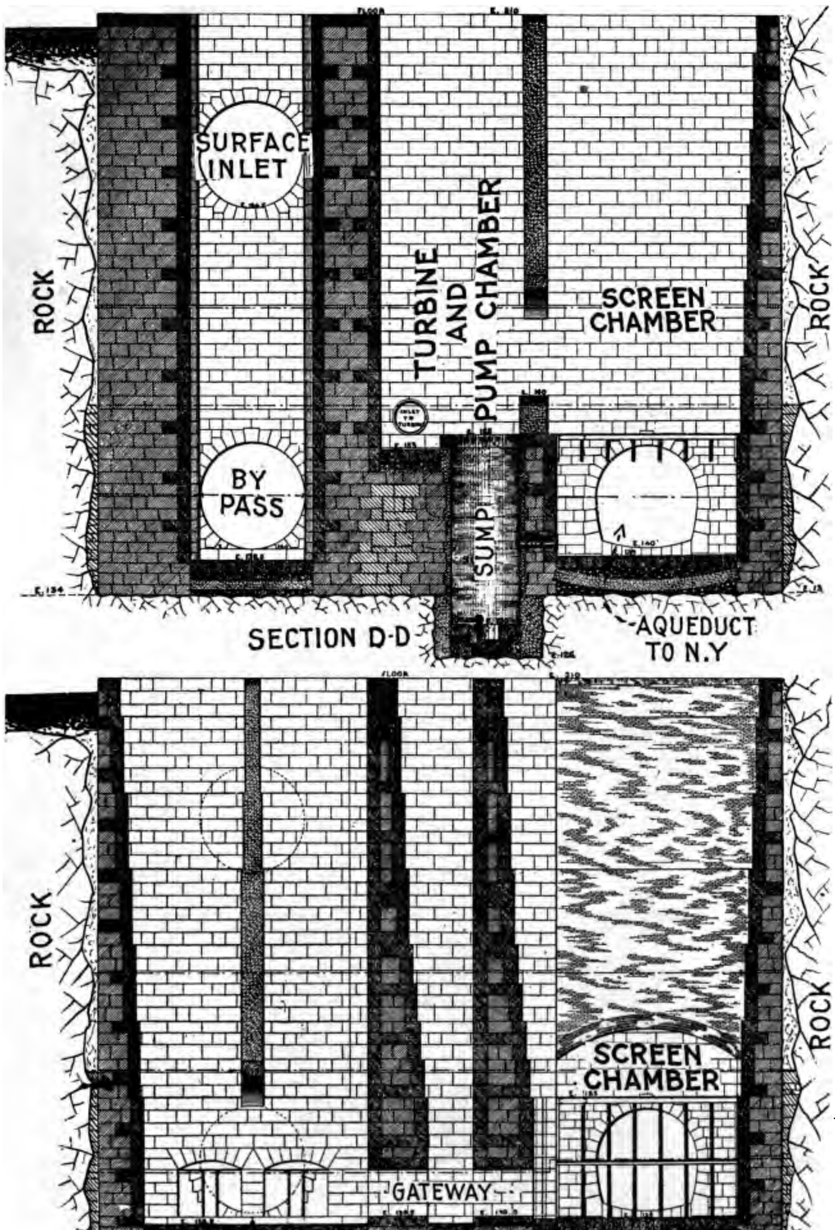
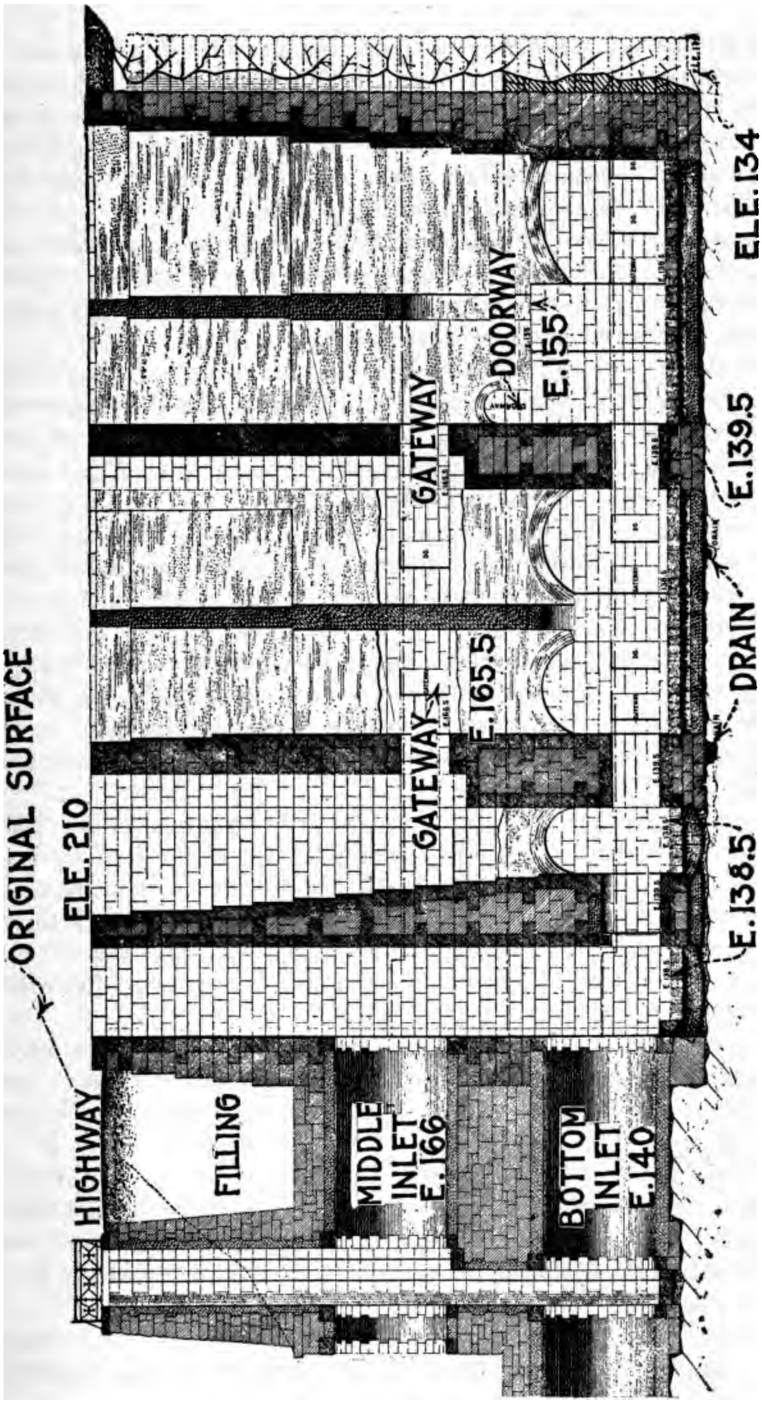


FIG. 177b.—Inlet Gate-house of New Croton Aqueduct.



SECTION F-F-F-F

FIG. 177c. —Inlet Gate-house of New Croton Aqueduct.

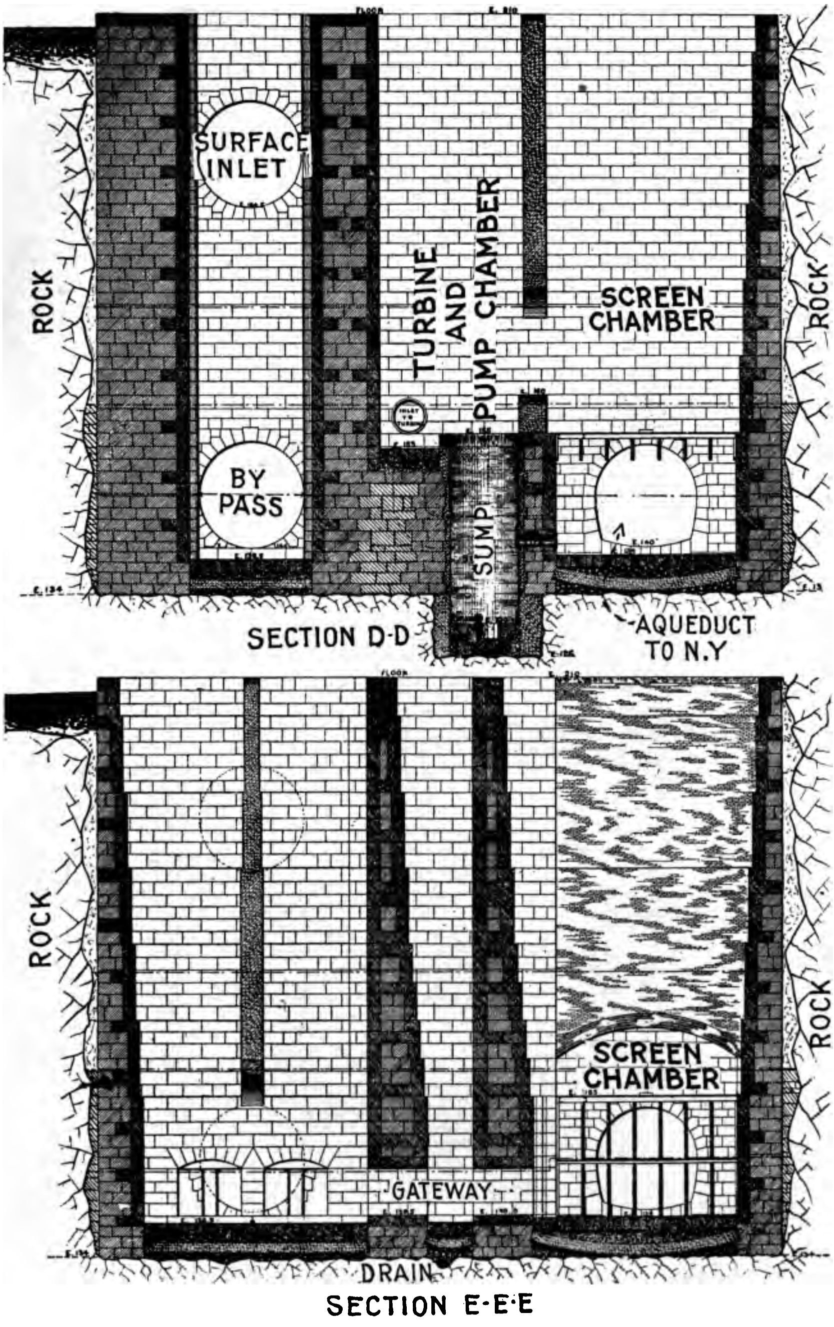
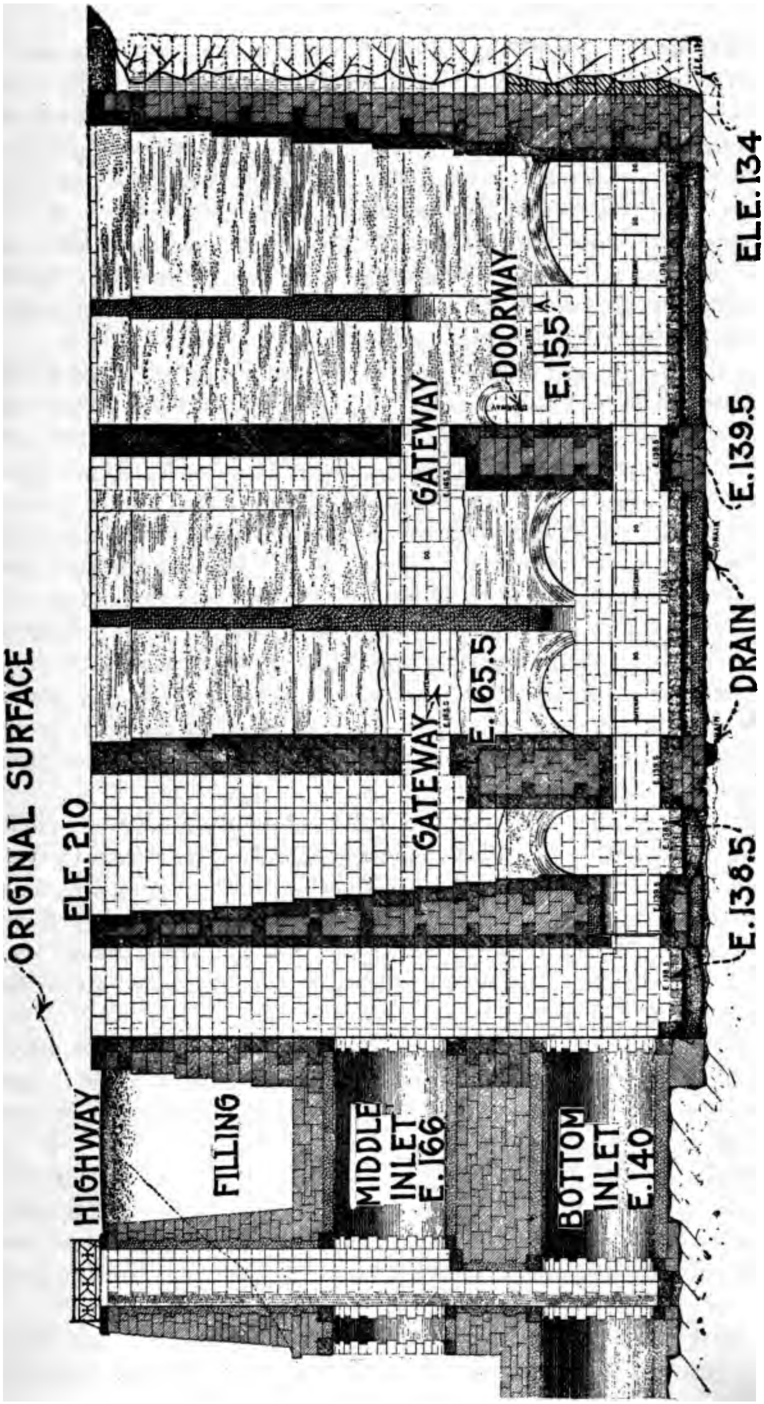


FIG. 177b. — Inlet Gate-house of New Croton Aqueduct.



SECTION F-F-F-F

FIG. 177*c*.—Inlet Gate-house of New Croton Aqueduct.

is built for the first 23 miles as a gravity conduit, the head of the water in the reservoir is gradually reduced in the gate-house by three or four sets of sluice-gates, placed in the cross-walls of the gate-house so as not to subject the aqueduct to pressure. The amount of opening of the different sets of sluice-gates is regulated so that the water will be about 8 or 9 ins. below the crown of the aqueduct at its inlet.

The main walls and principal cross-walls divide the substructure of the gate-house into ten water-chambers and a pump-chamber. Brick cross-walls, having arched openings, are built in some of the water chambers, to strengthen the principal walls.

Water flowing through any one of the inlets passes first into a small chamber which is gradually widened to make room for the first set of sluice-gates. At its entrance, 12 by 12 in. vertical grooves are cut in the side walls for stop-planks or drop-gates, made of wood and iron. From the first chamber, the water flows through sluice-gates into a second chamber, and then through another set of sluice-gates to the main water-chamber, which occupies a space of 28 by 30 ft. From here the water passes through a third set of sluice-gates into a large chamber on the northwest corner of the building, where a curved wall turns the water at right angles with the former direction. The water flows next through a fourth set of sluice-gates into the screen-chamber, from which it passes through vertical screens, made of No. 10 brass wire netting ($\frac{1}{4}$-in. mesh or opening) and fastened by brass screws to 4 by 8 ft. oak frames, into the aqueduct.

The screen-chamber has, also, a second set of sluice-gates at right angles to the first set. By this arrangement, water drawn from the by-pass or surface inlet can flow into the screen-chamber without passing through the main water-chamber. In this case, however, only three sets of sluice-gates are available for reducing the head of the water. By means of stop-planks, the screen-chamber can be separated from the adjoining water-chambers.

In order to relieve the pressure on the first set of sluice-gates, when the gate-house is empty, 12-in. iron pipes, provided with stop-gates, are placed in the first cross-walls of each inlet, 32.5 ft. below the flow-line of the reservoir.

Thirty-eight 3- by 6-ft. sluice-gates are required for the gate-house, which controls not only the flow into the new aqueduct, but can, also, regulate the flow into the old aqueduct. The latter can, however, also draw a supply through a small gate-house, built on the new Croton dam, about 3 miles further down-stream.

The water chambers are paved with granite blocks, and a drainage system is provided for emptying the gate-house. The drainage water is pumped out of a sump into the aqueduct by a 9-in. anti-friction centrifugal pump, operated by $30\frac{1}{2}$-in. turbine. Iron ladders are attached to the side walls of most of the water-chambers.

277. *The aqueduct* is constructed from the inlet gate-house to a terminal gate-house on Manhattan Island, New York — a distance of 32 miles — entirely in tunnel, with the exception of about 5900 lineal ft. of conduit, which was built in trenches. For about 25 miles from the inlet gate-house, the aqueduct is built as a gravity conduit with a horseshoe cross-section (Fig. 177*d*) and a grade of 0.7 ft. per mile, with the exception of a stretch of 1135 ft. under a swamp where the aqueduct is built as a pressure-tunnel, 14 ft. in diameter. This part of the aqueduct has a capacity of 300,000,000 gals. per day. It is connected with a large

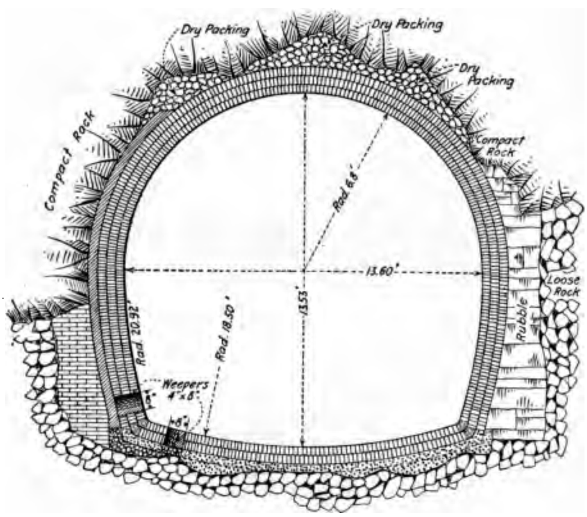


FIG. 177*d*. — New Croton Aqueduct (Gravity-tunnel).

receiving reservoir, known as the Jerome Park reservoir, which was built in the northern part of the city.

From this reservoir to the terminus on Manhattan Island — a distance of 6.94 miles — the aqueduct is constructed as a pressure tunnel (Fig. 177*e*), $12\frac{1}{4}$ ft. in diameter, except under the Harlem River, where the diameter is reduced to $10\frac{1}{2}$ ft. The capacity of this part of the aqueduct is 250,000,000 gals. per day.

The aqueduct tunnel was driven from 40 shafts and 2 inclines. It was lined throughout with brickwork, backed by rubble masonry or concrete. In the part that is not under pressure, weepers admit the ground water, which adds about 4,000,000 gals. per day to the supply drawn from the inlet gate-house.

Three blow-off gate-houses, each having an overflow-weir for regulating the depth of the water in the aqueduct, were built at water courses that crossed the line of the aqueduct.

278. *The terminal gate-house* (Figs. 177*f* and 177*g*) of the aqueduct at 135th Street and Convent Ave., in the city of New York, consists of a masonry substructure, containing the water-chambers, sluice-gates, etc., and of a fine masonry superstructure. The main building, which is about 53 by 79 ft. in plan, connects the new aqueduct with eight lines of 48-in. mains, which are joined to the distributing pipe system as explained below. An annex on the west side of the building is connected by a short masonry conduit with the old Croton

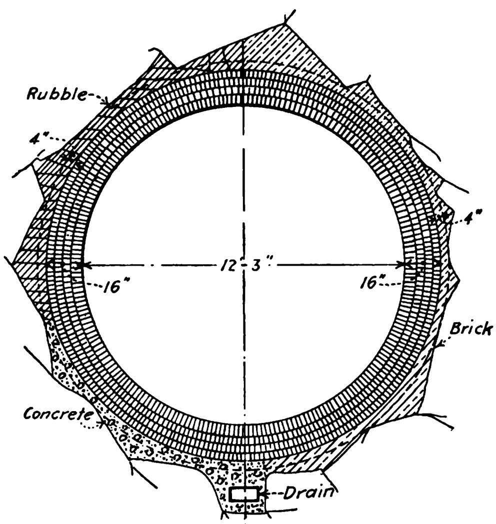


FIG. 177*e*. — New Croton Aqueduct (Pressure-tunnel).

aqueduct, which is near the gate-house, and by four 48-in. mains with the distributing pipes.

The main water chamber is 69.5 ft. long, 14 ft. wide, and 43.5 ft. deep, below the floor of the building. It is connected with the new aqueduct by a brick-lined well, $12\frac{1}{4}$ ft. in diameter and about 40 ft. deep, sunk at the northeast corner of the gate-house. The junction of the tunnel and the well is rounded off by a *goose-neck* turn. A central granite pier, about 12 ft. long, 3 ft. wide, and $43\frac{1}{2}$ ft. high, divides the inlet from the well into two water passages, which may be shut off by stop-planks, for which grooves are provided in the masonry. Seven piers, built of granite dimension-stone on the south side of the water-chamber, form eight small chambers, to each of which one of the water pipes is connected.

PLATE VI.

FIG. 1. — Terminal Gate-house of New Croton Aqueduct.

FIG. 2. — Pipe-line of New Croton Aqueduct.
(8 lines of 48-inch cast iron pipes.)

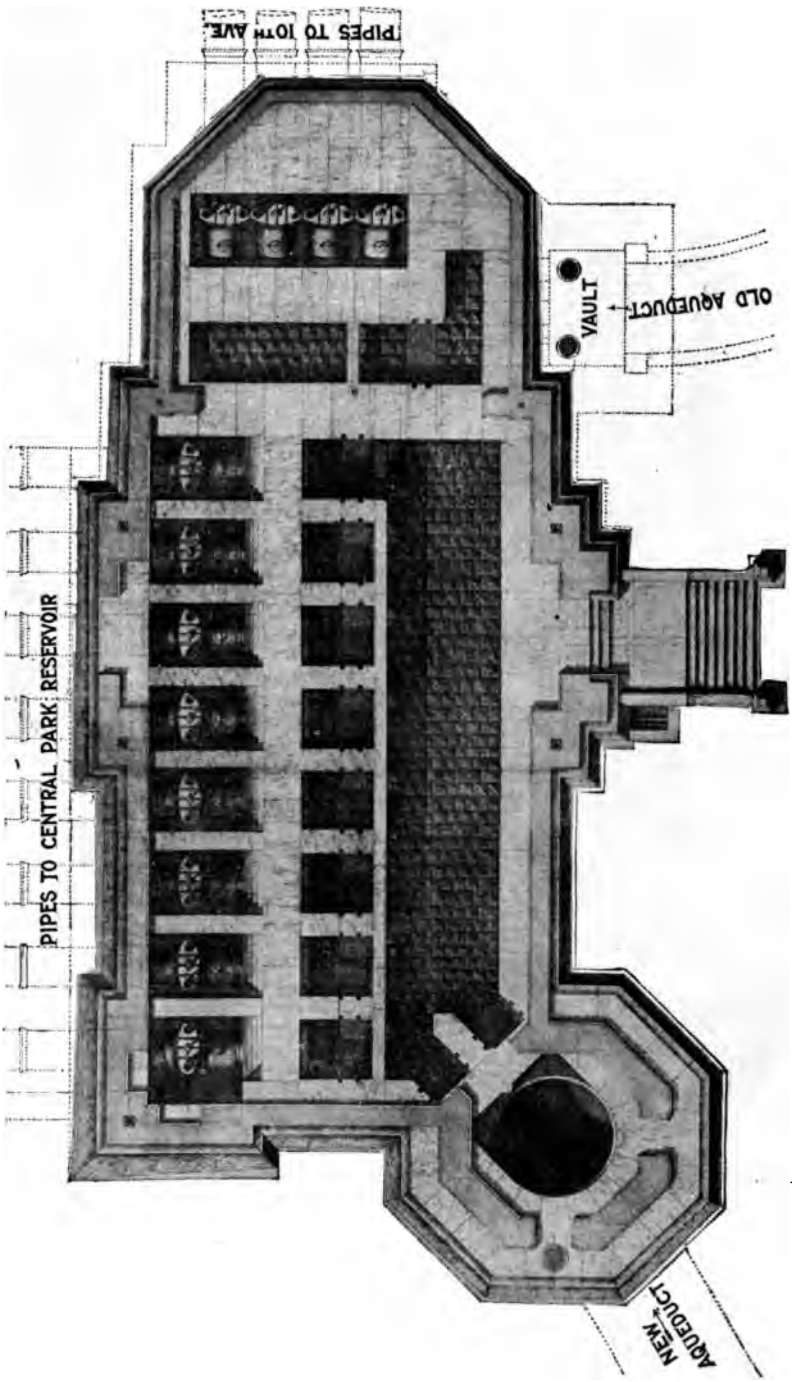


Fig. 177f.—Plan of Terminal Gate-house of New Croton Aqueduct.

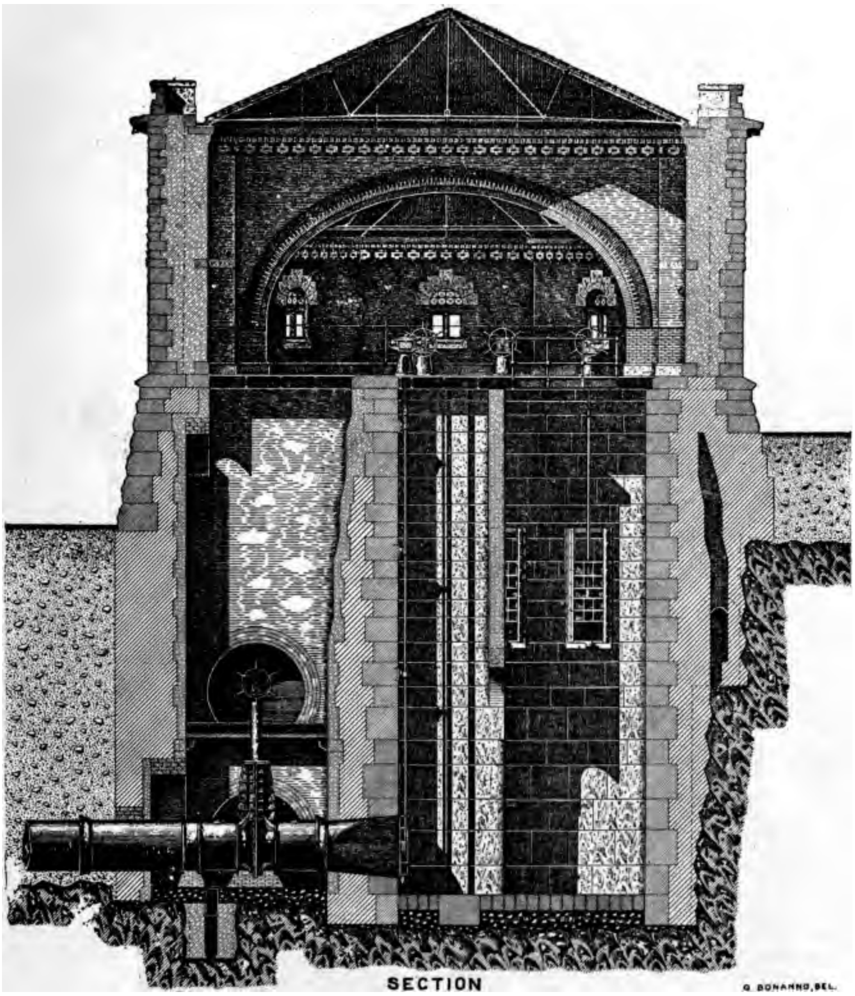


FIG. 177*g*.—Section of Terminal Gate-house of New Croton Aqueduct.

The piers are $2\frac{1}{2}$ ft. wide, 9 ft. long, and are spaced $6\frac{1}{2}$ ft. apart, in the clear. They are carried up from the paving of the water-chamber to the floor of the building. The north ends of the piers are joined by semi-circular brick arches, upon which a brick wall, 16 ins. thick, is carried up to the floor of the building. Two sets of grooves are cut on each face of each pier, and corresponding grooves are cut in the walls of the building opposite the end piers. One set of grooves is used for screens, and the other for stop-planks. The screens have 3- by 4-in. oak frames, fastened together by brass screws, and covered with wire netting made of No. 10 brass wires, spaced $\frac{1}{4}$ in. apart. The stop-planks are made of 6- by 12-in. yellow pine pieces, the ends being trimmed to fit the grooves (p. 247).

The pipe chambers are closed on the south side by a wall, 6 ft. wide at the base and 4 ft. wide at the top, which separates the water chamber from a vault in which a gate-valve is provided for each of the eight lines of water mains. Cast iron reducers, placed in the wall and connected with the gate-valves, form the inlets of the pipe-lines. A 2- by 5-ft. sluice-gate, set in front of each reducer, and operated by means of a hand-wheel or capstan bars from the floor of the building, controls the flow of water into the water main. From the above description, it will be seen that three means of shutting off the water are provided for each pipe-line, *viz.:* a sluice-gate, a gate-valve, and stop-planks.

The eight water mains laid in connection with the new Croton aqueduct were placed at a considerable depth at the gate-house, so as to be able to drain the receiving reservoir at Jerome Park, mentioned above. As the old water pipes are only about 4 ft. below the surface of the street, the water chamber in the annex, which is 31 ft. long and $6\frac{1}{2}$ ft. wide, was built at a higher elevation than the main water chamber. The arrangements in the former chamber as regards sluice-gates, gate-valves, etc., are exactly like those in the latter. Both chambers are paved with granite blocks, 18 ins. deep, placed on a concrete foundation, and are faced with granite masonry. Cast iron ladders, attached to the side walls, give access to the bottom of the water chambers, when the water is shut off. The gate-house can be drained by means of a sewer, which was built for this purpose.

279. *Over-flow Weirs and Blow-offs.* — At three places between the Croton reservoir and Harlem River, gate-houses were constructed, each having an overflow-weir for regulating the height of the water in the aqueduct, and blow-off gates for emptying the conduits. These gate-houses were all constructed on the same general plan (Fig. 178). Each has a substructure consisting of a masonry chamber, 37 by 55 ft. in the clear, and about 20 ft. high, which is divided by an overflow-weir,

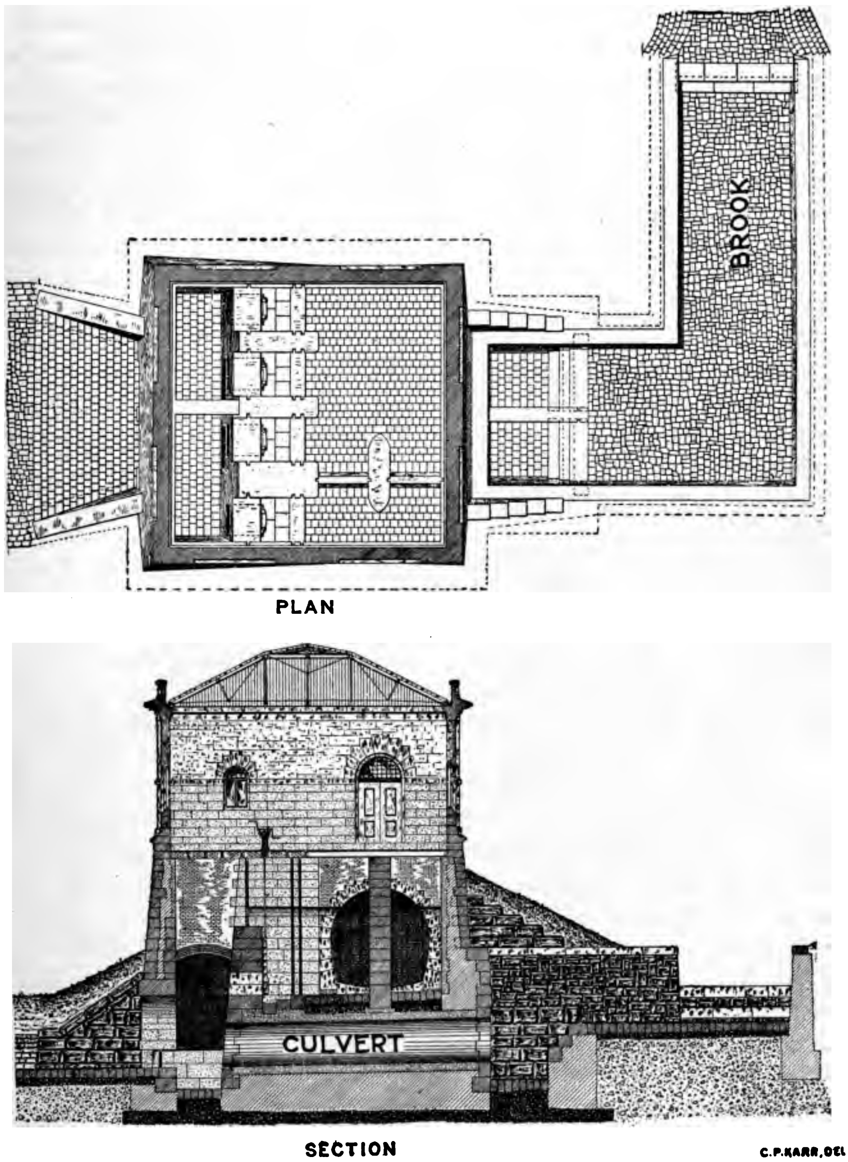


Fig. 178.—Overflow and Blow-off Gate-house of New Croton Aqueduct at South Yonkers, N. Y.

built parallel with the axis of the aqueduct, into two parts: a water chamber, 17 ft. wide, forming part of the aqueduct, and a waste-chamber, about 9 ft. wide, which receives the water flowing over the weir or discharged by the blow-off gates, and delivers it through a culvert to a brook or stream nearby.

The walls of the substructure consist of rubble masonry, faced with 8 or 12 ins. of brickwork. The overflow-weirs are constructed of granite cut-stone masonry. Three piers divide the overflow-weir into four parts, each 6 ft. wide. In each of these parts, an opening for a 3- by 4-ft. blow-off sluice-gate is provided. Three sets of 4½- by 4½-in. grooves for stop-planks are cut in the piers, and opposite to them in the north and south walls of the substructure. The first set of grooves is used for stop-planks, which are placed on top of the overflow-weir, for regulating the height of the water in the aqueduct. The other two are used for forming a dam of stop-planks in front of the blow-off gates, when required for repairs. The stop-planks are made of 6- by 12-in. yellow pine, dressed at the ends to fit the grooves. When they are placed in the grooves, a piece of tarred cord (marline) is generally tacked to the bottom side of each stop-plank, in order to make the joints between the planks as water-tight as possible.

Near the southerly end of the water-chamber, a central pier, 3 ft. wide by 11 ft. long, having both ends rounded, is placed in the middle of the water-channel, and divides the stream into two currents, each 7 ft. wide. Grooves, 4½ by 4½ in., are cut in the sides of this pier, in the east wall of the substructure, and in one of the overflow-piers, for stop-planks or timber drop-gates, by means of which a dam can be quickly formed across the water-channel. In this manner, the section of the aqueduct between two of the blow-off gate-houses can be separated from the other parts of the conduit, and emptied by the blow-off gates, when necessary for repairs or inspection. Each of the cast iron sluice-gates for the blow-off is 4 ft. wide and has two stems.

280. *Pipe-line* (Plate VI, Fig. 2). — Of the eight lines of 48-inch mains which form part of the aqueduct, four are laid from the 135th Street gate-house to a large receiving reservoir in Central Park — a distance of 2.38 miles. A small gate-house, built at the reservoir, controls the outlet of these pipes. The other four mains are laid from the terminal gate-house at 135th Street to different points where they are connected directly to the distributing system. At the lowest point of the pipe-line, blow-off gate-valves, placed in a masonry vault, are connected with the different lines of pipes.

281. *Distribution of Available Head.* — The total amount of available head for the aqueduct when the reservoir is full, is 33.70 ft. This is distributed as follows, the capacity of the gravity conduit being 300,000,000 gals. per day, and that of the other part of the aqueduct being reduced to 250,000,000 gals.:

Part of aqueduct	Length in miles	Head in feet
Gravity conduit	24.82	17.37
Pressure conduit	7.17	6.33
Loss on curves, etc.	....	1.50
Pipe-line	2.38	8.50
	34.37	33.70

As the pipe-line was the most expensive part of the aqueduct, it was given relatively more head than the other portions of the conduit.

282. *Engineers.* — The new Croton aqueduct was designed and constructed under the direction of the Aqueduct Commission of the city of New York. B. S. Church was chief engineer from 1883 to 1888, when he was succeeded by Alphonse Fteley, under whose direction the aqueduct was completed.

283. The Catskill Water Works * were constructed in 1906–1918 to obtain a supply of water from the Catskill Mountains for the city of New York. The water-sheds from which the city is authorized to draw water are: Esopus, 257 sq. miles; Schoharie, 314 sq. miles; Rondout, 143 sq. miles; and Catskill, 200 sq. miles, including several small contiguous areas. These water-sheds aggregate an area of 914 square miles, from which a minimum of 760,000,000 gals. can be obtained even in a series of extraordinarily dry years. The region is sparsely settled and furnishes a very pure supply of soft water.

The Esopus water-shed has been developed by the construction of the Ashokan reservoir, and the Catskill aqueduct was built to convey water from this reservoir to the city. A second impounding reservoir is soon to be constructed on the Schoharie Creek, and is to be connected with the Esopus Valley by a tunnel, 11 ft. 3 ins. high, 10 ft. 3 ins. wide, and about 18 miles long.

284. The Ashokan reservoir (Fig. 179) on Esopus Creek is the first impounding reservoir constructed in the Catskill Mountains water-sheds. The reservoir was formed by building across Esopus Creek at Olive bridge a masonry dam, 1000 ft. long, with a maximum height of 240 ft. above the foundation, which is flanked on both sides by earthen dams, and by constructing earthen dikes across smaller streams and at depres-

* Annual Reports of the Board of Water Supply, city of New York, 1906–17. Figs. 176 to 193 are reproduced from cuts given in these reports.

sions in the hills bounding the reservoir. It has an available storage capacity of about 128,000,000,000 gals. A *dividing dike and weir*, 2200 ft. long, divide the reservoir into an *east basin* and a *west basin* which have their flow-lines respectively at Elevations * 587 and 590. The overflow of the reservoir is discharged over a waste-weir, 1000 ft. long, on the boundary of the east basin, into a small affluent of Esopus Creek.

285. *Outlet-channels.* — In order to be able to draw down the reservoir to the desired depth, a deep channel had to be excavated in each of the basins to a point on the dividing dike where the outlet gate-chamber was constructed. The east channel is 3000 ft. long, and is excavated in rock to Elevation 500 with a minimum width of 26 ft.,

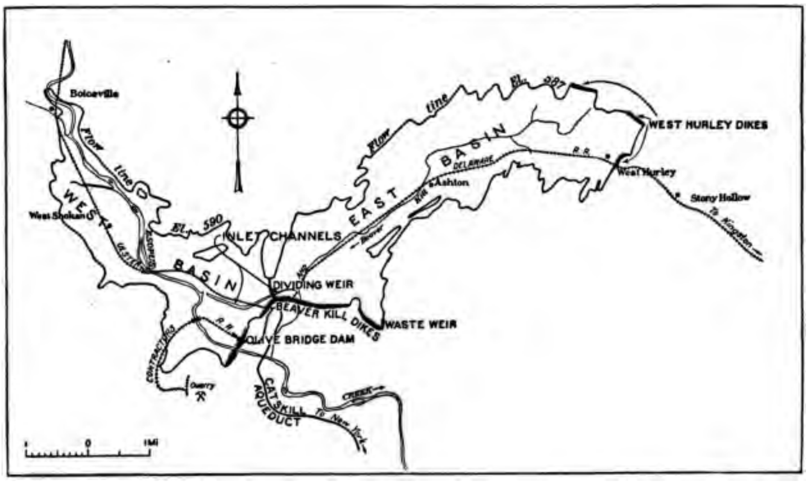


FIG. 179. — Ashokan Reservoir.

and slopes of 6 to 1. The west channel begins at the Esopus Creek. It is 5800 ft. long, and is excavated, mainly in earth, to a bottom elevation of 494 ft. with a width of 40 ft. and slopes of $2\frac{1}{2}$ to 1.

286. Headworks. — The flow of water from the reservoir into the aqueduct is controlled by means of an *upper* and a *lower gate-chamber*, provided with the necessary sluice-gates, valves, etc. From the lower gate-chamber the water can pass directly to a screen-chamber built at the head of the aqueduct; or it may be conveyed first to an aerator, where the water can be exposed in numerous small jets to the air, to purify it to a certain extent, and then be delivered to the screen-chamber. The two gate-chambers, the aerator and screen-chamber constitute the headworks of the aqueduct (Fig. 180).

* The elevations in this description refer to sea-level.

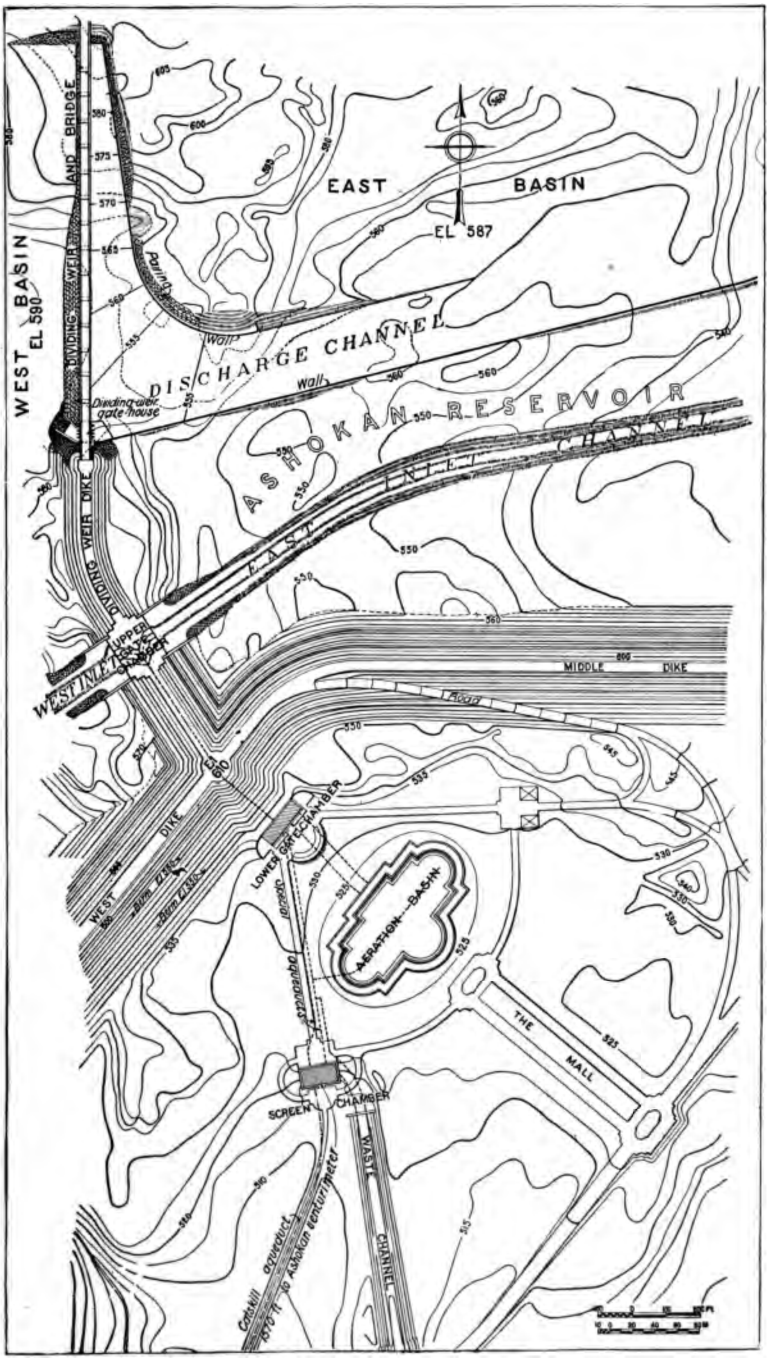


FIG. 180. — Headworks of Catskill Aqueduct.

287. *The upper gate-chamber* (Figs. 181 and 182) is located in the dike portion of the dividing weir on ground that is only about 15 ft. below the flow-line of the reservoir. As all the water drawn from the reservoir has to pass through this chamber, the excavation for the construction had to be made to a depth of about 80 ft. The chamber has at each of the two outlet-channels of the reservoir, twenty 6- by 12-ft. inlet-openings, placed in five rows of four openings each, at different levels. Coarse racks of bronze are placed in front of the inlet-openings, to prevent large objects from getting into the aqueducts. Back of the openings, cast iron grooves are provided in the concrete side walls for stop-planks or shutters, by means of which the depth at which water is drawn from the reservoir can be regulated.

After passing the stop-plank grooves, the water flows through short steel tubes, 60 ins. in diameter, one placed in the bottom of each inlet passage. Each of the steel tubes has a sluice-gate at its inlet, a gate-valve at its center, and a stop-disc, for emergency use, at its outlet. From the steel pipes, the water flows into one of the two pressure aqueducts leading to the lower gate-chamber. One of these aqueducts is built directly above the other. The upper one receives water from the east basin, while the lower one draws water from the west basin. The lower pressure aqueduct has a horse-shoe cross-section, $11\frac{1}{2}$ ft. high by 11 ft. 5 ins. wide, its invert being at Elevation 492. The upper pressure aqueduct has an oval cross-section, $9\frac{1}{2}$ ft. high by 14 ft. wide. Each of these aqueducts is designed to resist a maximum head of about 100 ft., and the two together are calculated to be able to deliver the water that can be drawn from the reservoir with all the gates wide open. The pressure aqueducts are about 650 ft. long, and lead to the lower gate-chamber where the quantity of water drawn from the reservoir is controlled.

The function of the upper gate-chamber is to regulate the depth at which water is taken from the reservoir, and it is to be used only to a limited extent, when the reservoir is first put into service, in controlling the volume of water drawn from the reservoirs.

288. *The lower gate-chamber* (Figs. 183 and 184) is constructed near the down-stream toe of one of the earthen dikes. It receives water from the reservoir through the two pressure aqueducts, and discharges it into two special aqueducts leading to the screen-chamber of the main aqueduct; or through other outlets to the aerator. The two special aqueducts, which are about 700 ft. long, are built for the greater part in the same trench, but one of them is placed lower than the other, so that it can be used for emptying either of the basins of the reservoir. It is provided with a 6 by 15 ft. waste-gate at the screen-chamber. The outlet of the

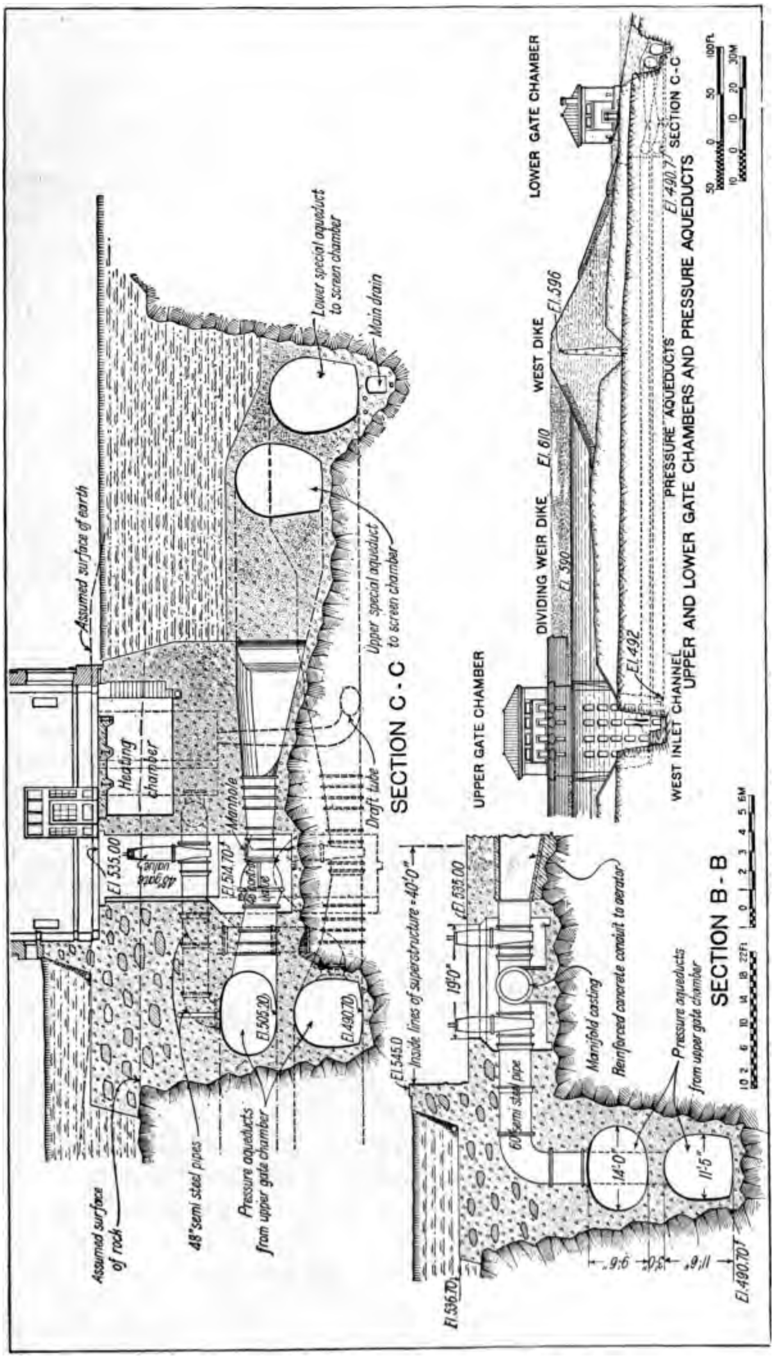


FIG. 181. — Upper Gate-house of Catskill Aqueduct.

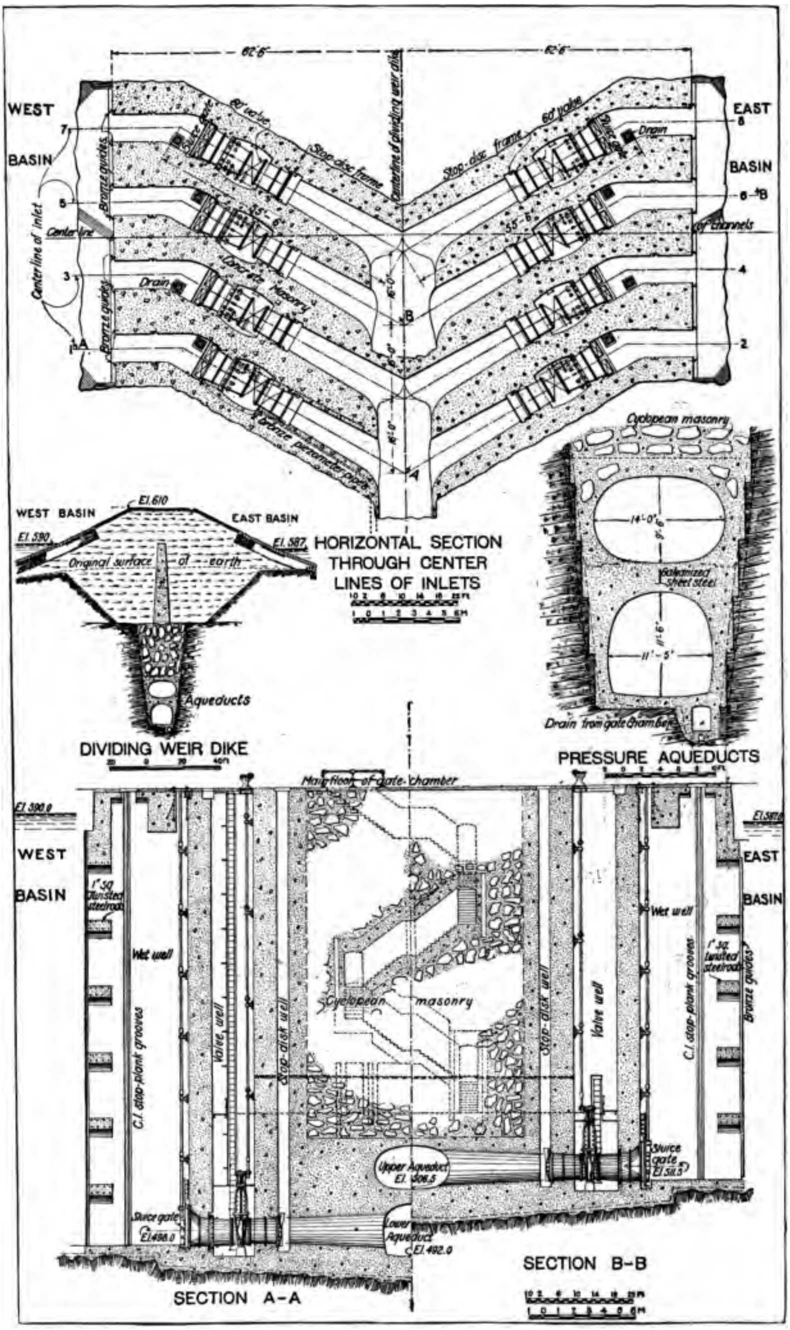


Fig. 182. — Upper Gate-house of Catskill Aqueduct.

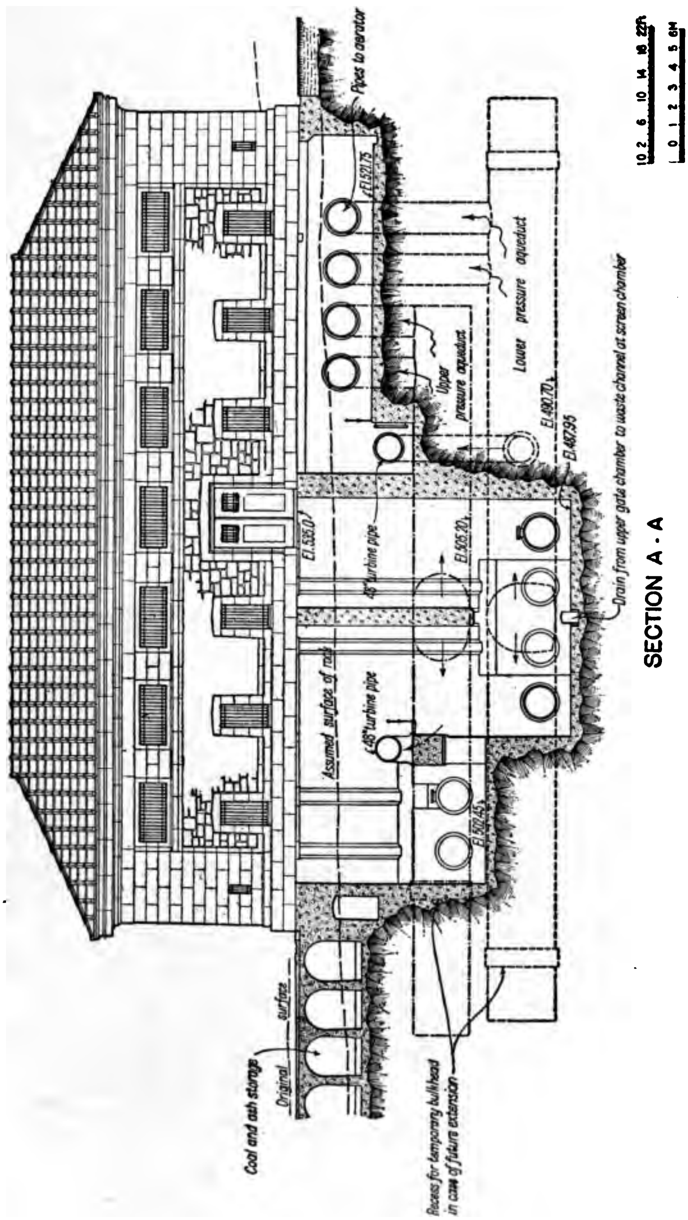


FIG. 183. — Lower Gate-house of Ashokan Reservoir.

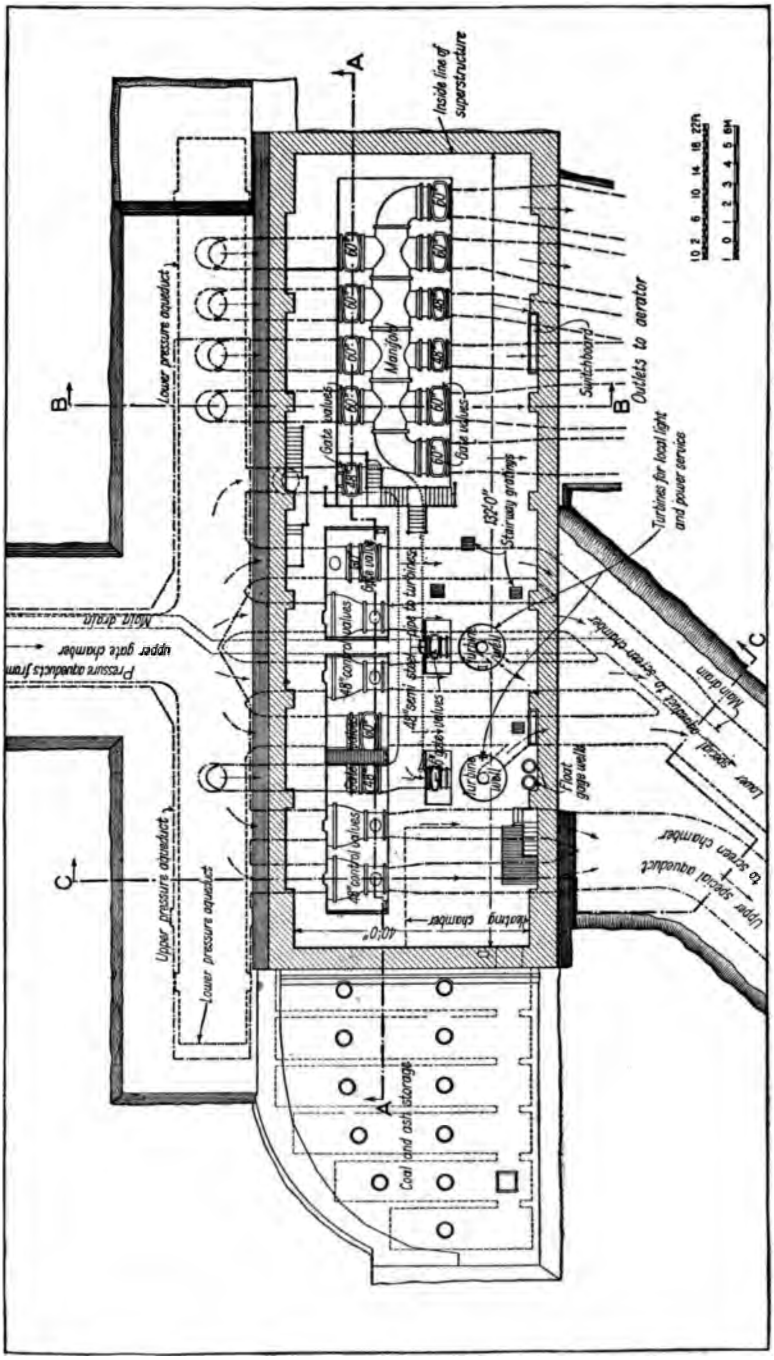


FIG. 184. — Plan of Lower Gate-house of Ashokan Reservoir.

aerator is connected with the upper special aqueduct, so that aerated water can be delivered to the aqueduct, while bad water is being wasted through the lower special aqueduct. The water area of the special aqueducts is approximately a mean between that of the pressure aqueducts and the standard cut-and-cover section of the aqueduct. The two pressure aqueducts are connected in the gate-chamber by ten 60-inch steel pipes with the two special aqueducts and with reinforced concrete conduits leading to the aerator in the following manner: The lower pressure aqueduct is connected with the lower special aqueduct by four horizontal pipes, and with the aerator conduits by two vertical pipes. The upper pressure aqueduct is connected with the upper special aqueduct by two horizontal pipes and with the aerator conduit by two vertical pipes. The flow through each steel pipe is controlled either by a 60-in. gate-valve, or by a special 48-in. control-valve, designed to avoid the *chattering* that occurs frequently when water passes through large gate-valves or sluice-gates. The four vertical pipes supplying water to the aerator are connected with a manifold out of which lead four 60-in. and two 48-in. pipes to the aerator conduits. The object of the manifold is to reduce the pressure of the water supplied to the aerator, when necessary. The head that is available at the lower gate-house, when the reservoir is not drawn down too low, is utilized, by means of two turbines, to supply power for operating the gates, lighting the gate-house and vicinity, etc. Each of the pressure aqueducts supplies water to one of the turbines through a 48-in. steel pipe, controlled by a gate-valve. Two float-wells, connected by bronze gauge pipes with the main aqueduct below the screen-chamber, are provided in the gate-chamber, in order to indicate, at all times, the level of the water at the inlet of the Catskill aqueduct. A suitable gate-house is built over each of the gate-chambers.

The two special aqueducts, leading to the screen-chamber, are constructed in the same trench until near the screen-chamber. They reach the hydraulic grade (Elevation 510) just above the screen-chamber. In order to make it impossible to subject the main aqueduct, at any time, to a pressure, an overflow weir, 100 ft. wide, is constructed in the lower special aqueduct, just above the screen-chamber, with the crest at the hydraulic grade-line. The upper special aqueduct is provided with an overflow-weir, 40 ft. wide, with its crest at the same level as that of the lower special aqueduct. The two waste-weirs together are calculated to be able to discharge any excess of water above the capacity of the main aqueduct, that might be drawn from the reservoir by having all gates wide open. An ample waste-channel, covered for 100 ft. at the screen-chamber, discharges into Beaverkill gorge.

289. *The screen-chamber* (Figs. 185 and 186) is made sufficiently large to serve for two aqueducts. It is divided into two symmetrical parts by piers having grooves for stop-planks. The Catskill aqueduct is connected with one of these divisions, and the other has an outlet, now closed with a masonry bulkhead, for some future aqueduct. Either division, or both, can be used for screening the water. In order to economize space, the grooves for the screens have been arranged so that the screens can be placed in a broken line, with a view of providing the required screen area in a minimum of space. This arrangement facilitates, also, the removal of the screens, and makes it possible to handle three screens in one operation. Grooves for guard gates, to be used when the main screens are being cleaned, are placed immediately below the main screens. Float-wells are provided in this chamber to show the loss of head caused by the screens. Stop-planks and screens not in use are stored in a room built for this purpose, from which a track extends to the main floor. Water under pressure from the reservoir can be used for washing the screens on a washing floor, at one side of the screen-chamber.

290. *Capacity of the Headworks.* — The headworks are planned to make it possible to draw 500,000,000 gals. per day, from either basin, through the lower special aqueduct into the Catskill aqueduct, when the water surface of the reservoir is as low as Elevation 518. If water is taken from both basins, this quantity can be drawn when the surface of the reservoir is at Elevation 516. On account of the throttling caused by the two 48-in. control-valves, the upper special aqueduct can only deliver 500,000,000 gals. per day when the water in the reservoir is at Elevation 540. With all valves wide open, the two pressure aqueducts can draw 500,000,000 gals. per day with the reservoir surface at Elevation 514. The discharges mentioned above are all for direct delivery to the main aqueduct without the water passing through the aerator.

The aerator basin is of irregular shape, about 240 ft. wide by 460 ft. long. Reinforced concrete conduits connect it with the lower gate-chamber. About 1600 bronze nozzles discharge the whole flow of the aqueduct in a series of jets, to enable the oxygen of the air to free the water of objectionable odors and gases. A horse-shoe shape aqueduct, having a side-slit on top to receive the aerated water, passes through this basin, and discharges over a weir into the upper special aqueduct, just above the screen-chamber.

291. The Catskill Aqueduct (Figs. 187 and 188), built in 1907 to 1917, has a maximum capacity of over 500,000,000 gals. per day. It has a length of about 92 miles from the inlet gate-house at the Ashokan reservoir to the Hill View reservoir, an equalizing basin of 900,000,000

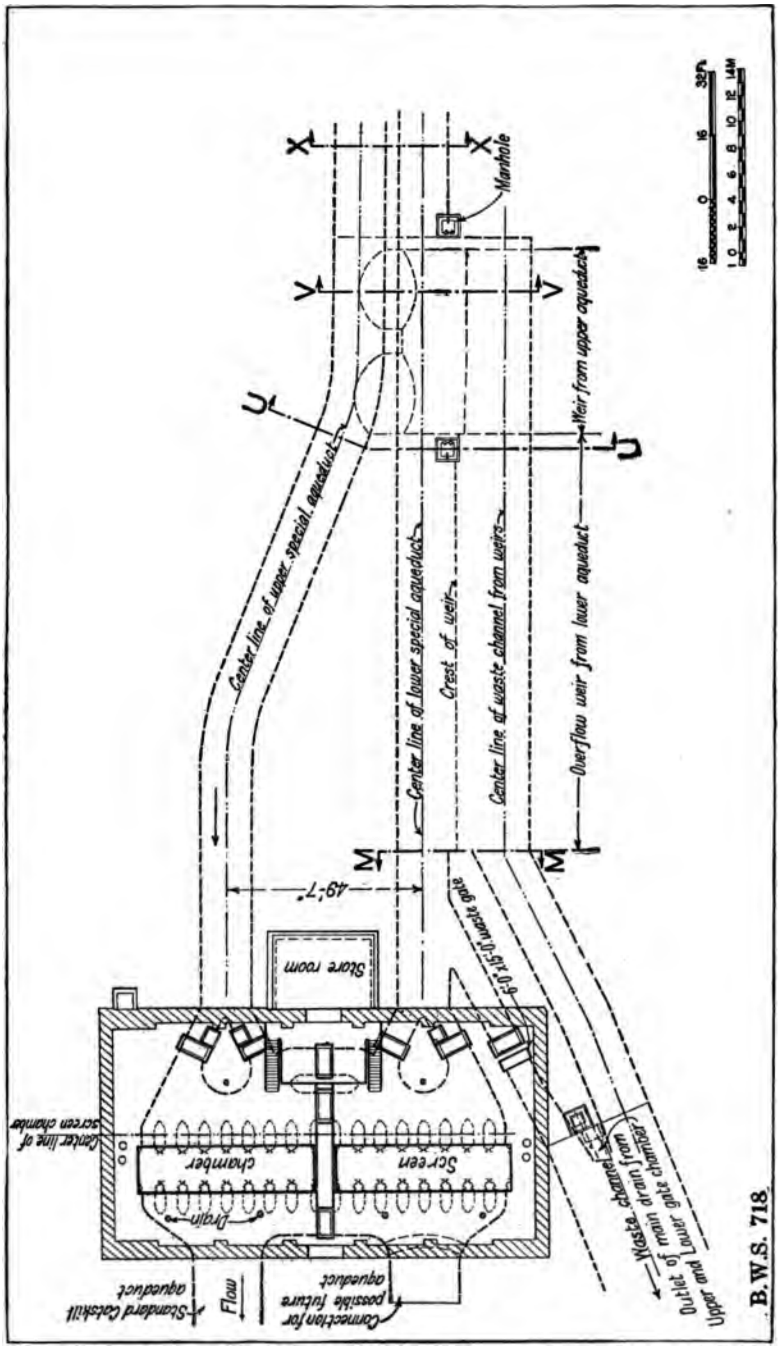


FIG. 185. — Screen-chamber of Catskill Aqueduct.

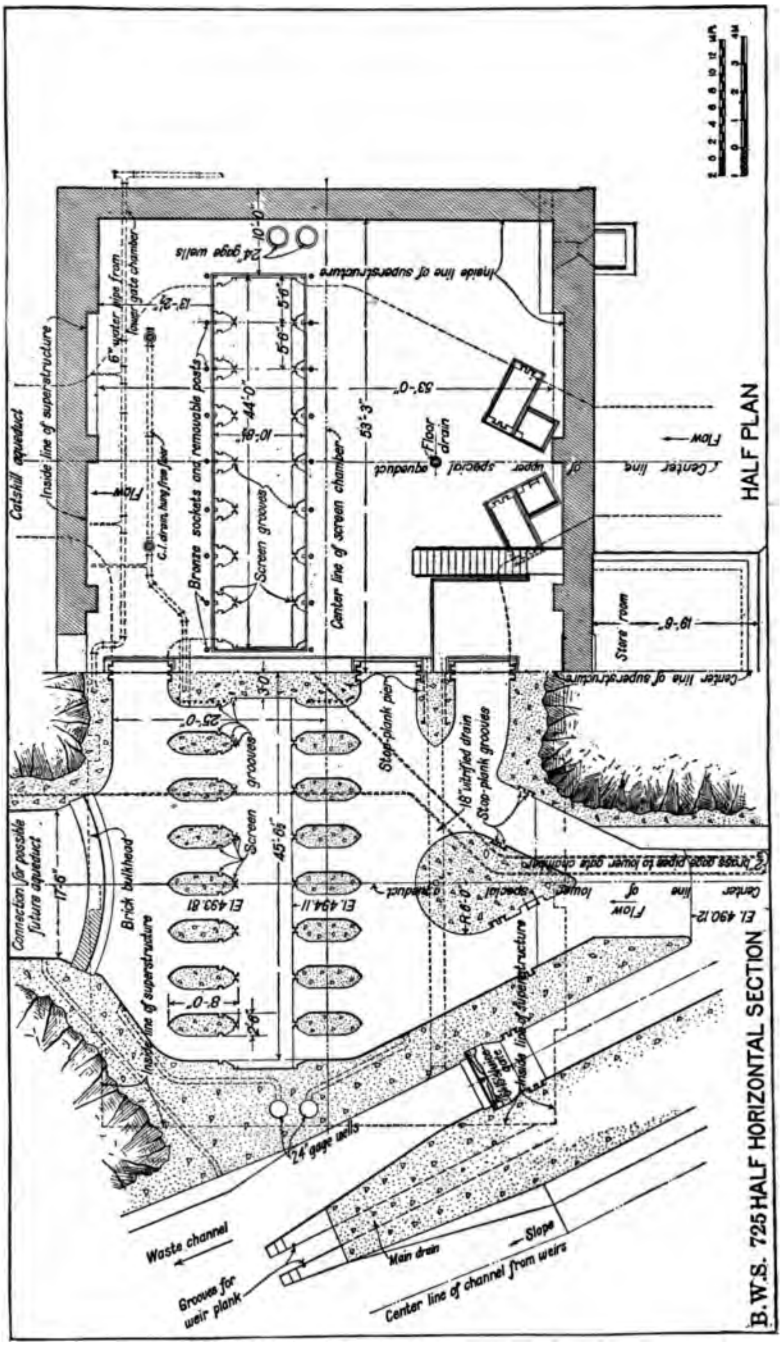


Fig. 186. — Screen-chamber of Catskill Aqueduct.

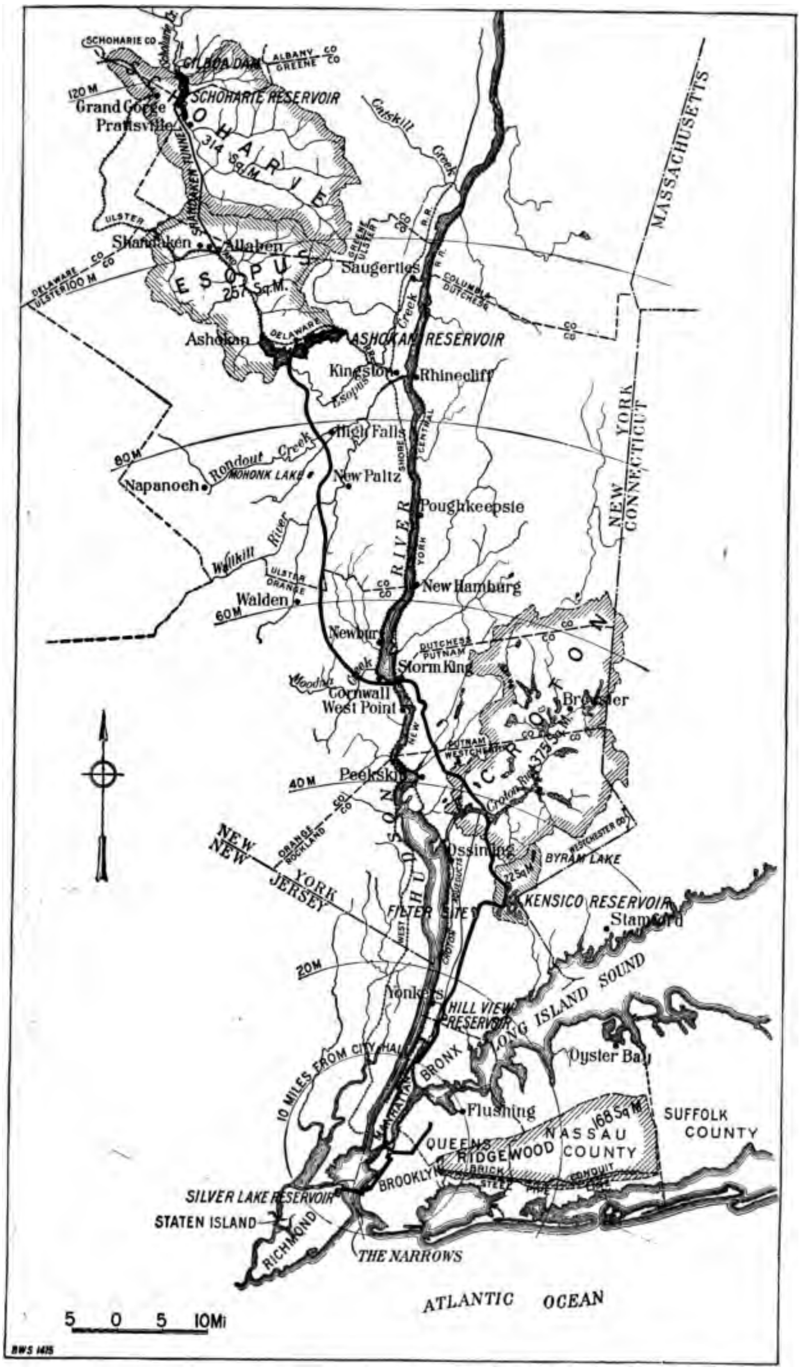


Fig. 187. — Map of Catskill Aqueduct.

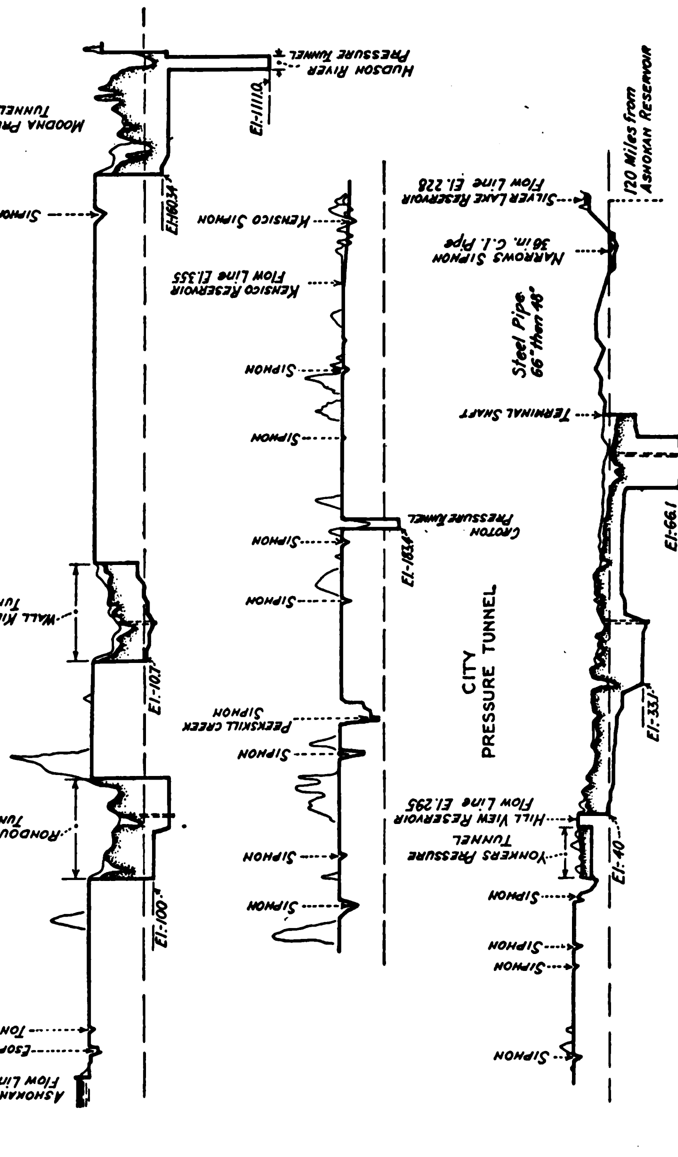


Fig. 188. — Profile of Catskill Aqueduct.

gals. capacity, constructed near the north boundary line of the city of New York. This reservoir was formed by excavation and embankment, and is divided by a central wall into two basins. Its flow line is at Elevation 295.

From the Hill View reservoir, the water is distributed to the five boroughs of the city by a circular tunnel, called the *City Tunnel,* which is driven 250–750 ft. below the surface of the ground, through solid rock. Its diameter is 15 ft. at the reservoir and is reduced, a foot at a time, to 11 ft. at the end of the tunnel in Brooklyn. The City tunnel, which is 18 miles long, terminates at two shafts from which the water is distributed by steel and cast iron pipes. The water is taken across the Narrows, the entrance to the harbor of New York, in a line of 36-in. cast iron pipes with flexible joints (p. 159), which is continued by a 48-in. main to a terminal reservoir on Staten Island, called Silver Lake, storing about 435,000,000 gals. Measuring along the line of the aqueduct, the distance from the Ashokan reservoir to Silver Lake is about 120 miles. It takes the water about three days to traverse the distance.

The main aqueduct from the inlet to the equalizing reservoir is composed of the following parts:

	Miles
Cut-and-cover work	55
Grade tunnels	14
Pressure tunnels	17
Pipe-siphons	6
Total	92

The types of construction adopted for the different parts of the aqueduct are shown on Fig. 189.

As the cut-and-cover work was the cheapest of these types, it was adopted where possible. For this type, the aqueduct is given a horseshoe cross-section, 17 ft. high by 17½ ft. wide, and is covered by an earth embankment. Where hills or mountains cross the line of the aqueduct, and cut-and-cover work would have lengthened the line too much, grade-tunnels were driven. Twenty-four tunnels of this type, generally 17 ft. high by 13⅓ ft. wide, were constructed. To reduce the cost, their cross-section was given a smaller area than that of the cut-and-cover work, and the gradient was made steeper. All of the tunnels are lined with concrete. Weep-holes are generally left in the lining, to admit the ground water, and to relieve the external pressure [illegible] are emptied. Where the ground water is of unde[illegible] holes are omitted, and the lining is r[illegible] external pressure.

292. *Pressure-tunnels.* — W[illegible] phons, consisting either of p[illegible]

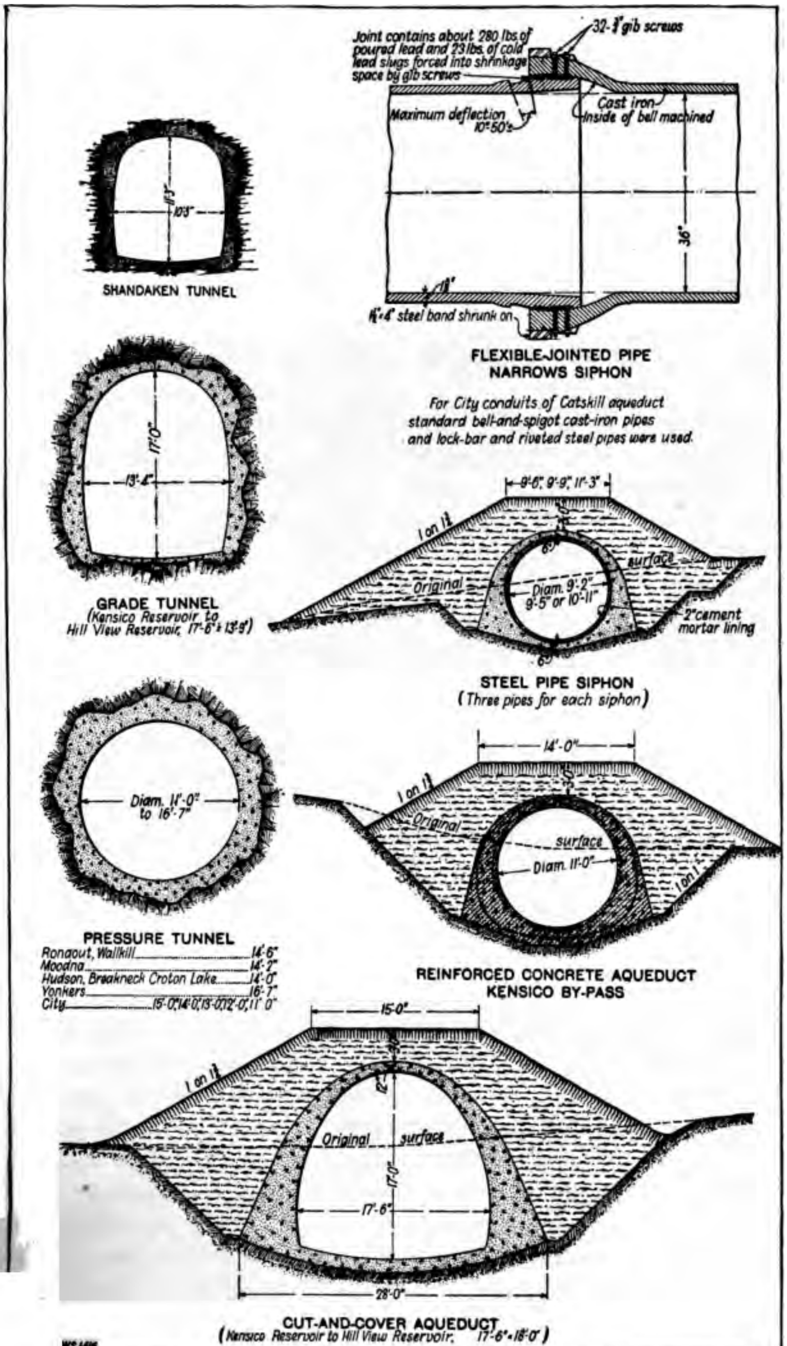


Fig. 189. — Typical Cross-sections of Catskill Aqueduct.

type, being the cheaper of the two, was adopted where a thickness of at least 150 ft. of solid rock over the tunnel could be obtained. Seven pressure-tunnels, with a diameter of about 14½ ft., were constructed. At each end, each pressure-tunnel, with one exception, is connected with the adjoining cut-and-cover work by a circular shaft. The exception occurs where a pressure-tunnel joins a steel-pipe siphon. For the purpose of unwatering the pressure-tunnel, drainage shafts were constructed between the end shafts of the tunnels, usually near some large stream. Where the tunnels cannot be emptied by gravity, floating pumps are used.

The most important pressure-tunnel was driven through granitic rock, under the Hudson River, at a depth of 1114 ft. below sea-level. It is 14 ft. in diameter and about 3000 ft. long (Fig. 190). The top of the west shaft is closed by a thick layer of concrete. The east shaft, which, besides serving as a water-way and access shaft, is also used as the drainage shaft of an adjoining pressure-tunnel, has a removable cover made of steel castings and forgings. The shaft has an inner diameter of 14 ft. A steel lining, 15 ft. in diameter, is placed in the concrete-lining, to insure water-tightness. About 10 ft. above sea-level, the shaft is covered by a steel casting, which is nearly hemispherical in shape. This dome rests upon a cast steel ring or curb, and is kept in place against a head of about 410 ft., to which it is subjected when the aqueduct is in service, by 36 anchor bolts of nickel-chrome steel, each 4½ ins. in diameter and 50 ft. long. These bolts pass through holes, bored in the flange of the dome and in the curb, and through steel sleeves to an anchor-ring of cast steel, which is placed in the shaft, 46 ft. below the curb.

293. *Pipe Siphons.* — There are fourteen steel-pipe siphons on the line of the aqueduct, each of which is to consist of three lines of pipe. The diameters of the pipes in the different siphons vary from 9½ to 11¼ ft. The pipes are of steel, $\frac{7}{16}$ to ¾ ins. thick, according to the pressure to be resisted. Alternate sections are made small enough in diameter to fit inside of adjoining sections. The circular seams and joints of the pipes are single or double riveted, according to the pressure on the siphons. All longitudinal seams are triple riveted (Fig. 190*a*). The rivets are all 1 in. in diameter. No coating, except whitewash, was used, as the pipes were to be protected by cement mortar. Owing to a steeper gradient when in partial operation, each pipe-line has half the capacity of the masonry conduit, *viz.*, 250,000,000 gals. per day. Only one line of pipe was laid at each siphon as a first instalment. The other lines are to be laid when needed.

The pipes are lined on the inside with a 2-in. coating of 1 to 2 Portland

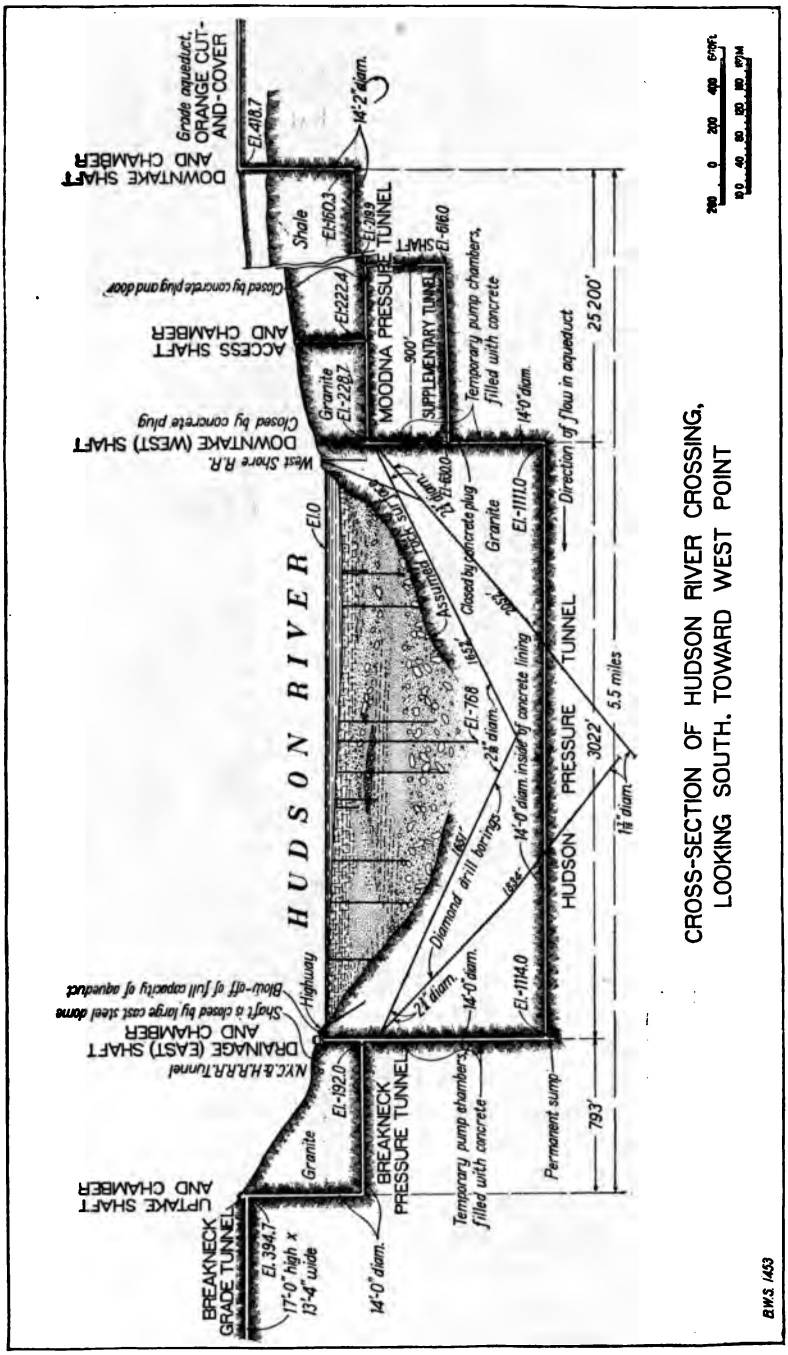
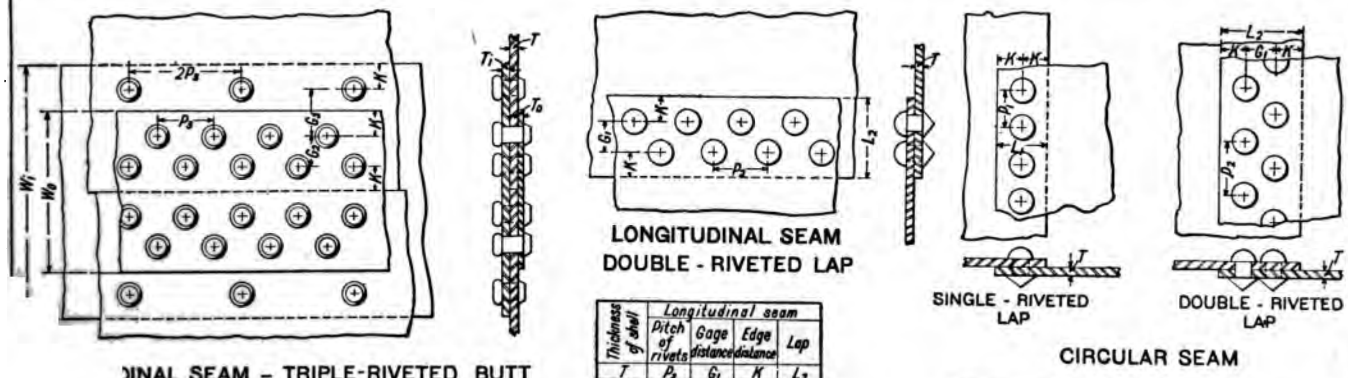


FIG. 190. — Pressure-tunnel across Hudson River.



Thickness of shell	Longitudinal seam: Pitch of rivets	Gage distance	Edge distance	Lap
T	P_2	G_1	K	L_2
7/16"	3.50"	1 15/16"	1 5/8"	5 3/16"
1/2"	3.11"	2"	1 5/8"	5 1/4"

Longitudinal seam: …ch of rivets, …ner …ws	Outer rows	Gage distance: Inner rows	Outer rows	Edge distance	Width of cover plates: Outer	Inner	Thickness of cover plates: Outer	Inner
P_3	$2P_3$	G_2	G_3	K	W_o	W_i	T_o	T_i
…58"	7.16"	1 15/16"	3 1/8"	1 5/8"	10 3/8"	16 5/8"	7/16"	9/16"
…00"	8.00"	2 1/16"	3 1/8"	1 5/8"	10 5/8"	16 7/8"	1/2"	5/8"
…82"	7.64"	2 1/16"	3 1/8"	1 5/8"	10 5/8"	16 7/8"	1/2"	5/8"
…58"	7.16"	1 15/16"	3 1/8"	1 5/8"	10 3/8"	16 5/8"	1/2"	5/8"

Assumed tensile stress on net section of plate 15000 lbs per sq. in.
" bearing " " rivets and plate 22500 " " " "
" shearing " " plate 10000 " " " "
" " " " rivets-single 9000 " " " "
" " " " " double 8000 " " " "
All rivets 1 inch in diameter before driving
All computations based on diameter of driven rivet — 1 1/16"

Thickness of shell	Edge distance	Single riveted: Number of rivets	Approx. pitch	Lap	Double riveted: No. of rivets each row	Approx. pitch	Gage distance	Lap
T	K		P_1	L_1		P_2	G_1	L_2
7/16"	1 5/8"	144	2.5"	3 1/4"	←103	3.50"	1 15/16"	5 3/16"←
1/2"	1 5/8"	not used			116	3.11"	2"	5 1/4"
9/16"	1 5/8"	"	"		123	2.95"	2 1/16"	5 5/16"
11/16"	1 5/8"	"	"		139	2.61"	2 3/16"	5 7/16"
3/4"	1 5/8"	"	"		145	2.50"	2 3/16"	5 7/16"

Double riveted joint with 7/16" plate is for use only at and 100' either side of the stream crossings at Indian brook and Tongore creek.

All pipe to have alternate inside and outside courses, i.e. not to taper

FIG. 190a. — Riveted Joints for Steel-pipe Siphons on Catskill Aqueduct.

cement mortar. On the outside the pipes are protected by 6 to 18 ins. of concrete, over which an earth embankment is placed. At the lowest point of the valley the pipes are supported by masonry abutments, between which sufficient room is left for the water course draining the area. Blow-offs are provided at these points.

The pipes are laid on concrete cradle-blocks, 3 ft. long, $3\frac{1}{2}$ ft. wide, and 6 ins. deep, placed $7\frac{1}{2}$ ft. from center to center. The blocks were kept slightly below grade until the pipes had been laid and wedged to the true grade, when the spaces between the cradles and the pipe were grouted. On steep slopes the cradle-blocks are kept in place by struts.

294. *The Kensico Reservoir.* — A large storage basin, called the Kensico reservoir, was constructed about 15 miles north of the boundary line of the city. It has an available capacity of 29,000,000,000 gals. — a quantity sufficient to supply the city for several weeks, should the Catskill aqueduct above this reservoir be shut off, for any cause. This reservoir was formed by building a masonry dam, having a total length of 1825 ft. and a maximum height of 307 ft. above the foundation, across the valley of the Bronx River. A depression in the hills, about a mile northwest from the dam, is filled by an earth embankment, about 1450 ft. long and 25 ft. high. The Catskill water is discharged into the reservoir at its upper end, where an influent-weir and gate-house is built. Water is drawn from the reservoir through a short tunnel at a point on the west side of the reservoir, about a mile above the main dam. An effluent gate-house, with sluice-gates for controlling the flow, is built at the reservoir side of the tunnel, and a large gate-house chamber is constructed at the outlet side of the tunnel. By means of sluice-gates in this chamber, the water flowing through the tunnel can be turned directly into the aqueduct or diverted through the Kensico aerator, which is built like the aerator at the headworks. The screen-chamber, in which all the water is passed through fine mesh screens before it is sent to the Hill View reservoir, is constructed near the large gate-chamber. A by-pass of concrete, 11 ft. in diameter and 11,000 ft. long, is built from the influent gate-house, to make it possible to convey the water directly to the city, without passing through the Kensico reservoir.

295. *The City tunnel* (Fig. 191) was driven from twenty-five shafts, twenty-two of which are connected with the present distribution system [illegible] the city. Thirteen of these shafts have single 48-in. risers each; [illegible] e two 48-in. risers each, and in each of the three other shafts are [illegible] in. risers. All of the shafts, excepting two, are circular, and [illegible] with concrete during the sinking. The upper 100 ft. of these

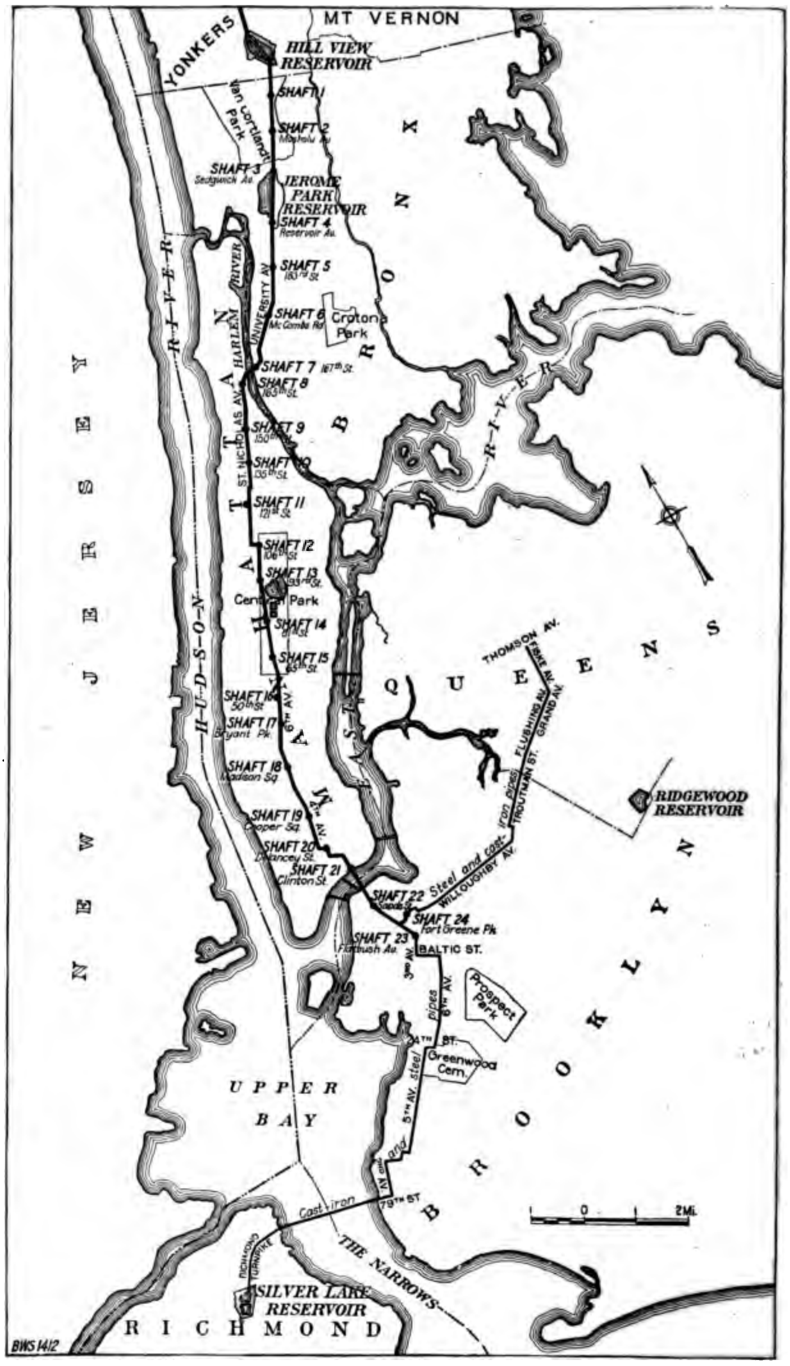


FIG. 191. — Plan of City Tunnel of Catskill Aqueduct.

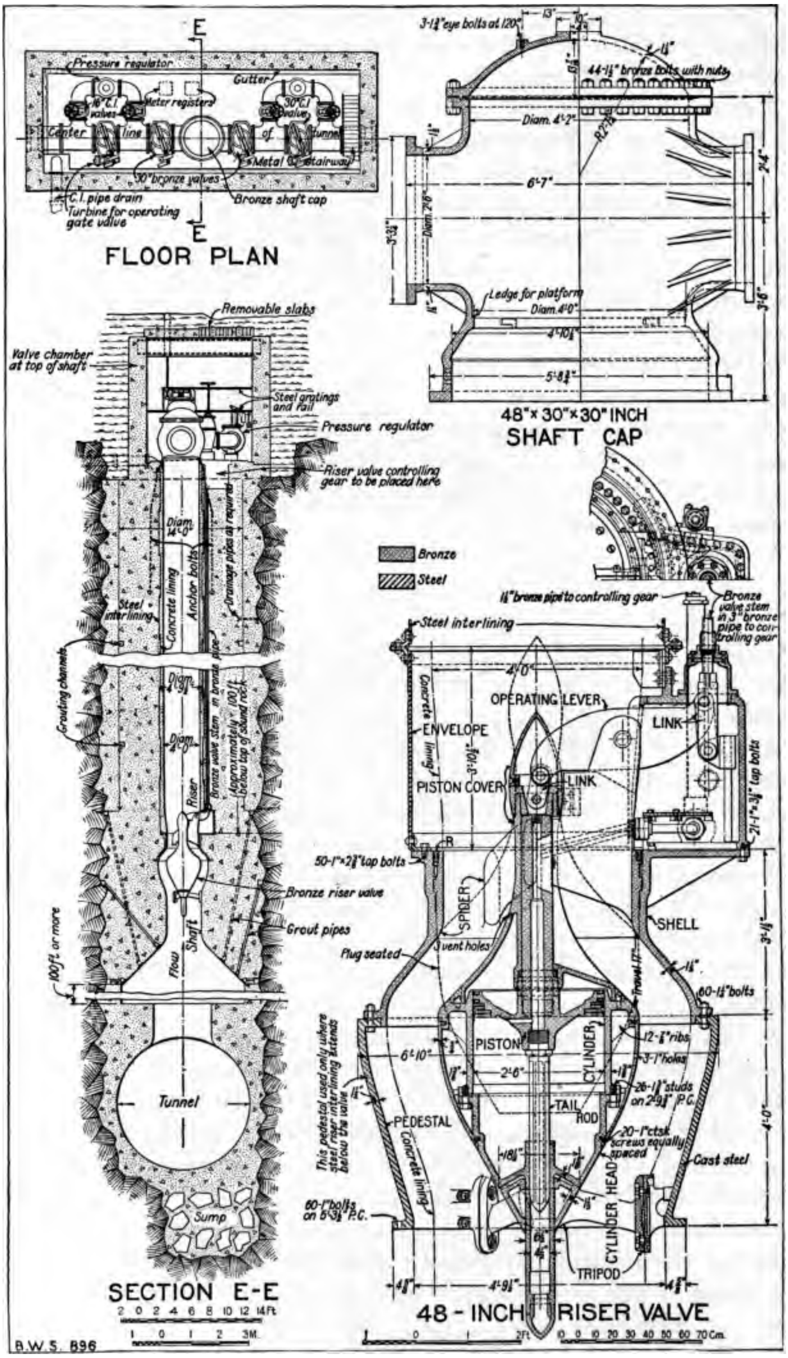


FIG. 191*a*. — City Shaft of Catskill Aqueduct.

shafts is provided with the riser pipes around which concrete is filled solid to the sides of the shafts.

A conical valve is attached to the bottom of each riser, and at its top the riser terminates in a bronze shaft-cap, which is practically either a 48- by 30-in. or 72- by 48-in. tee. From these caps the water passes through gates and regulating gates into the distributing pipes of the city. At two of the shafts, provision is made for unwatering the City tunnel, which is divided into three sections, each of which can be emptied for inspection, cleaning or repairs, independently of the others. This is accomplished by placing at each of two of the tunnel shafts (Nos. 13 and 18), a 66-in. section-valve on each side of which a bronze reducing pipe is placed, and connected to steel castings embedded in the concrete lining of the tunnel. On each side of the section valve the tunnel is made conical so as to reduce it gradually as it approaches the section-valve. Each of the valves is operated by a hydraulic cylinder, placed in the chamber at the head of the shaft.

At two of the shafts, the City tunnel is connected, respectively, to the new Croton aqueduct, and to the Jerome Park reservoir. At the top of each of the shafts at which the tunnel is connected with the distributing system, a chamber is constructed, to contain the valves and other appliances which are required to control the flow of the water. Bronze riser valves, 48 ins. and 72 ins. in diameter, are placed in shafts (Fig. 191*a*), about 100 ft. below the top of sound rock. In case of a break in the valve-chamber, or in the steel mains, causing an abnormally large flow of water, the riser-valves close automatically. The whole city tunnel is lined with concrete which was made water-tight by grouting under pressure. At one place where leakage occurs when the aqueduct is under full pressure, the interior surface of the conduit is lined, for a few hundred feet, with copper.

In the plans for the aqueduct, provision is made for constructing eventually a filter plant, about two miles below the Kensico reservoir. As a temporary measure, a small coagulating plant is built over the aqueduct, about two miles above the Kensico reservoir.

296. *Venturi Meters* (Fig. 192). — In order to measure the water delivered by the aqueduct, three Venturi meters — the largest ever made — are placed in the conduit as follows: one just below the Ashokan reservoir, one above, and another below the Kensico reservoir. Each of these meters is 410 ft. long, and is constructed of reinforced concrete, excepting the throat casting and piezometer ring, which are made of cast bronze. The former is 7¾ ft. in diameter and about 8 ft. long. The latter is 17½ ft. in diameter and 1 ft. wide. These two castings are set about 30 ft. apart. Besides the Venturi meters, five gauging chambers,

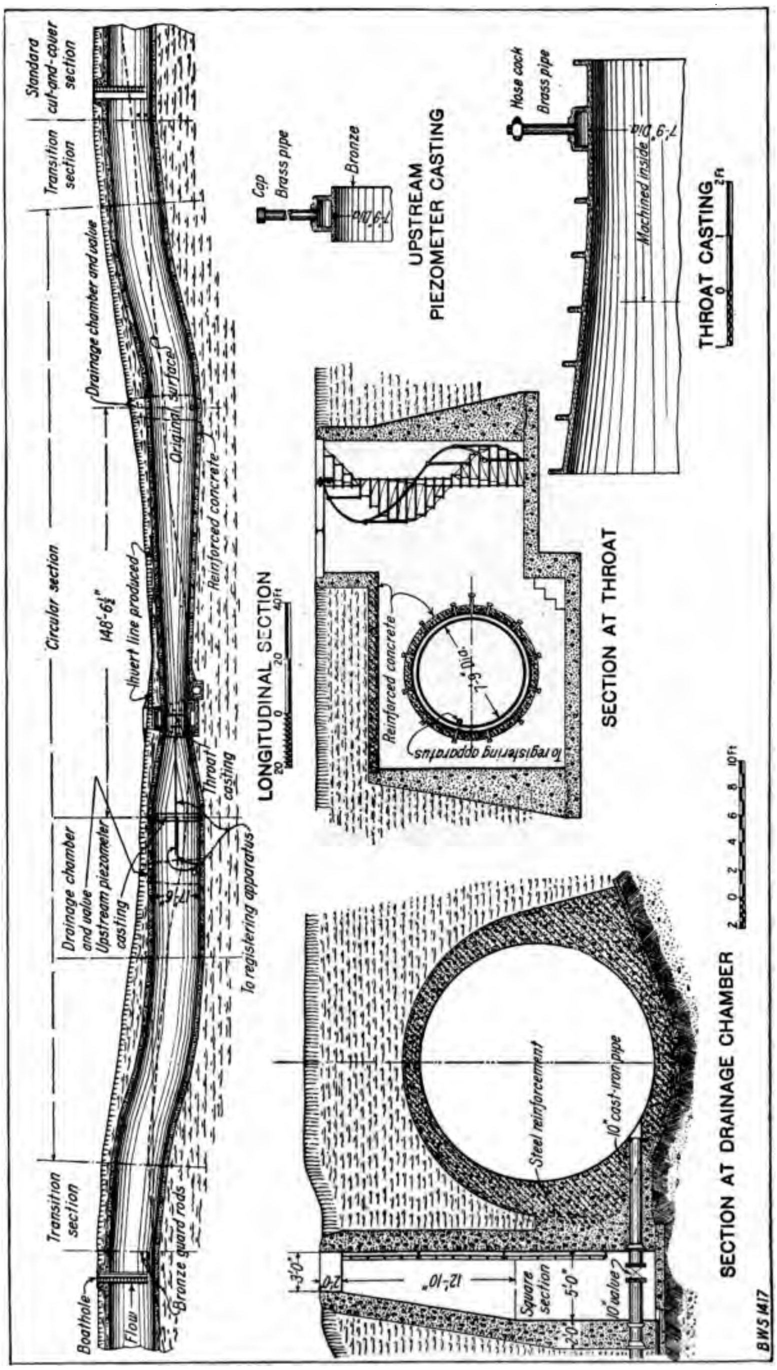


FIG. 192. — Venturi Meter at Ashokan Reservoir.

where the flow of the water can be measured by means of current meters, have been constructed at various points along the aqueduct. A Venturi meter is, also, placed in the city tunnel, near its beginning, and another one, capable of measuring flow in both directions, is installed at the connection with the Jerome Park reservoir. Provision has, also, been made for placing Venturi meters at each of the connections between the city tunnel and the distributing pipes.

297. *Engineers.* — The Catskill aqueduct and reservoirs were constructed under the direction of the Board of Water Supply of the City of New York, which appointed J. Waldo Smith as chief engineer and John R. Freeman, Prof. William H. Burr and Frederick P. Stearns as consulting engineers. Alfred Noble served also as consulting engineer of the Board from Sept. 16, 1909, to the time of his death, April 19, 1914.

The plans for the works were prepared by the Headquarters Department of which Alfred D. Flinn had charge until he was promoted on Aug. 19, 1917, to the position of deputy chief engineer.

298. Aqueducts of Boston, Mass. (Plate VII). — Boston built its first aqueduct in 1846–49, to obtain a supply of water from a chain of ponds, about $3\frac{1}{2}$ miles long, called Lake Cochituate. The drainage area of the lake contains 17.58 square miles. A dam was constructed across the outlet of the ponds to form a storage basin, and an aqueduct was built to convey the water to a distributing reservoir, located in the town of Brookline.

In 1873–79 the city obtained an additional supply from the Sudbury River, which has an available water-shed of 75.2 square miles. Four storage reservoirs were built in this water-shed, and a closed masonry conduit, called the Sudbury aqueduct, was constructed to convey the water to the city.

In 1895–1904 additional works were constructed to increase the water supply of the city, by obtaining a supply from the south branch of the Nashua River. By building a masonry dam across the river, a storage reservoir of a capacity of about 64,500,000,000 gallons was formed. A masonry conduit, called the Wachusett aqueduct, was built to convey the water from this reservoir to one of the reservoirs in the Sudbury water-shed, from which the water is taken by a second masonry conduit, called the Weston aqueduct, built in 1901–04, to the westerly limit of the Metropolitan District of Boston. Short descriptions of these aqueducts are given below.

299. *Engineers.* — The Sudbury aqueduct and reservoirs were designed and built under the direction of Joseph P. Davis, City Engineer of Boston. Mr. Alfonse Fteley was in immediate charge of the works as resident engineer.

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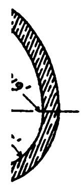

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The Wachusett and Weston aqueducts were built under the direction of the Metropolitan Water & Sewerage Board of Boston of which Frederick P. Stearns was chief engineer during the construction of these works.

300. *The Cochituate aqueduct** (Fig. 193) consists of an oval conduit of brickwork, 6⅓ ft. high, having a maximum width of 5 ft. It is 14.31 miles long, and has a fall of 4.26. The conduit is lined with brickwork, 8 ins. thick, which is plastered with cement mortar on the outside. The aqueduct was generally built in trenches. At two places, the conduit is built in tunnels. The masonry conduit is continued by a siphon consisting of 2 lines of 30-in. and one line of 36-in. cast iron pipes, which are carried across the Charles River on a masonry bridge of three arches. In 1875 a 40-in. pipe was added. The siphon is 1095 ft. long, and has a fall of 0.52 ft. Its greatest depression is 52.11 ft. below the level of the aqueduct. The aqueduct has four waste-weirs and one ventilator. Manholes admitting access to the conduit are built at intervals of ¼ mile, and are covered with stone slabs. The aqueduct empties into the Chestnut Hill reservoir. It has a capacity of 18,000,000 gallons per day, which can be increased to 23,000,000 gallons, by putting the conduit under pressure.

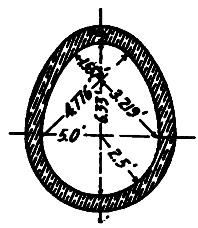


FIG. 193.—Cochituate Aqueduct. (Grade 1 : 5000.)

301. *The Sudbury aqueduct*† extends from Farm Pond, which served, until recently, as a storage reservoir, to the Chestnut Hill reservoir in Boston, a distance of 15.9 miles. The water in the available drainage area of the Sudbury River is brought to Farm Pond in a masonry conduit, called the Sudbury supply conduit, which extends from the lowest reservoir, built in connection with this aqueduct, to Farm Pond, a distance of about 7922 ft.

The main aqueduct and the supply conduit are both designed to have the same capacity. As the grade of the aqueduct is one foot per mile, while that of the conduit is 2.323 ft. per mile, the cross-section of the former is made larger than that of the latter. The maximum delivery of the aqueduct and conduit was fixed originally at 80,000,000 gallons per day, but was raised later to 108,000,000, by carrying a greater depth of water, in order to meet the increasing consumption.

* Reports of Water Commissioners of Boston, Mass., 1846–49; also "History of the Boston Water Works," compiled by a member of the Water Board, Boston, 1868.

† "Additional Supply from the Sudbury River for Boston, Mass.," by A. Fteley, Boston, 1882.

The aqueduct and supply conduit are built in a similar manner (Figs. 194 and 195). The foundation is of concrete, and the side walls are of rubble masonry, laid in cement mortar. The inner surface of the aqueduct has a brick lining, 4 inches deep, on the bottom and sides. The arch and its haunch backing are built of brickwork. In the standard cross-section, the arch is 12 ins. thick for an arc of 40 degrees from the spring-line, and is then reduced to a thickness of 8 ins. near the key. Where needed, in special cases, the thickness of the arch is increased. For the greater part, the aqueduct was constructed as *cut-and-cover work*, but in crossing some valleys it had to be built on embankments, the highest being 34 ft. above the natural surface. These fills were made of carefully selected earth, which was rolled in thin layers, and allowed to stand one season before being used. Where there was danger of settling, a thicker foundation course of concrete was used.

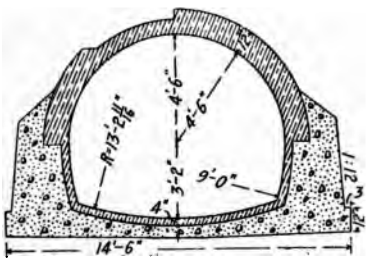


FIG. 194. — Main Sudbury Aqueduct. (Grade 1 : 2500.)

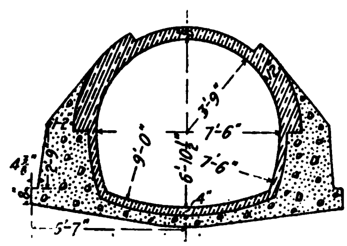


FIG. 195. — Sudbury Supply Aqueduct. (Grade 1 : 1250.)

The aqueduct is carried across Waban Brook by a masonry bridge, 536 ft. long, having 9 semicircular arches of 44⅔ ft. span. A second masonry bridge, 475 ft. long, carries the conduit across the Charles River. It has 7 arches, the largest of which has a clear span of 130 ft. (p. 251).

The flow through the Sudbury aqueduct is controlled in an inlet gate-house at Farm Pond. At four places on the aqueduct, waste-weirs and blow-off gate-houses were built. A terminal gate-house was constructed for the conduit between the two basins of the Chestnut Hill reservoirs. This gate-house has five outlet pipes (four, 48 ins. in diameter, and one, 60 ins. in diameter). Three of the mains empty into the Chestnut Hill reservoir, and the other two are connected directly to the distributing system.

302. *The Wachusett aqueduct* * (Fig. 196), which conveys water from the Wachusett reservoir to the Sudbury reservoir, is about 12 miles

* *Eng. Record*, Feb. 22, 1896, p. 205; May 9, 1896, p. 404; June 20, 1896, p. 47; May 27, 1899, p. 585.

long: For the first 2 miles, it is constructed in tunnel, partly in rock and partly in earth. For the next 7 miles, the aqueduct is a masonry conduit, built generally as cut-and-cover work, but carried in a few places by earth embankment. For the next 3 miles, the conduit is continued by an open channel, 20 ft. wide on the bottom, and having side slopes of 3 to 1, which were faced with gravel, where necessary, in order to prevent erosion. Two low dams are built across the open channel — one about midway of its length, and the other at its entrance into the reservoir — to keep back sand and gravel carried by the water. The grade of the tunnel section and open channel is 1 in 5000. The aqueduct has a horse-shoe cross-section, and can deliver 300,000,000 gallons per day.

The tunnel section has a brick invert throughout, in order to have a smooth floor. In hard rock, the arch and side walls are omitted, the

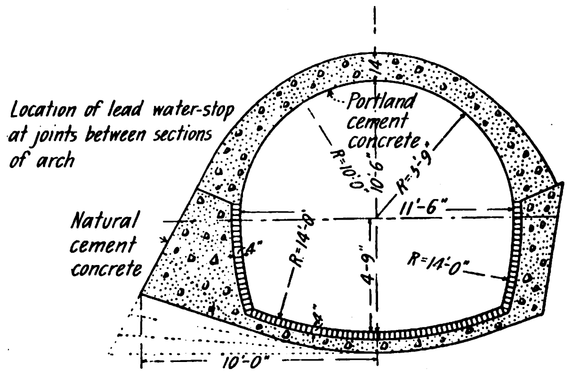


Fig. 196. — Wachusett Aqueduct.

tunnel being excavated so as to have a width of $13\frac{1}{2}$ ft. by a height of 11 ft. 10 ins. Where required, the tunnel is lined with brickwork backed with rubble masonry, the inside dimensions of the horse-shoe section being 12 ft. 2 ins. wide by $10\frac{1}{2}$ ft. high. The masonry conduit is built of Portland cement concrete, lined with 4 ins. of brickwork on the bottom and sides. It crosses the Assabet River on a masonry bridge, 359 ft. long, having seven arches of $29\frac{1}{2}$ ft. span (p. 251). The aqueduct has two granite buildings: one built to cover a gauging chamber, and the other over the terminal chamber of the masonry conduit.

At both the Sudbury and Wachusett reservoirs, the water is discharged through hydraulic turbines, and used for the generation of electric energy which is sold for industrial uses. This is said to be the first hydroelectric development in connection with a public water supply.

303. *The Weston aqueduct** begins at a head-house, built near the northerly end of the dam of the Sudbury reservoir, and extends to the bluff on the west side of the Charles River, a distance of about 13½ miles. It terminates in the town of Weston, at the westerly limit of the Metropolitan District, about 10 miles from the State House in Boston.

The head-house of the aqueduct is connected with the reservoir by three lines of 60-in. cast iron mains, each about 500 ft. long, which are controlled by valves at the dam. For the first 3½ miles, the aqueduct is built on a slope of 4 in 5000. It is then built, at a crossing under a railroad, of concrete and steel, according to a special plan. The grade of the aqueduct is then reduced to 1 in 5000, and its cross-section is increased (Fig. 197) for a distance of about 7 miles. The conduit is continued by an open channel, 1400 ft. long, to an equalizing reservoir

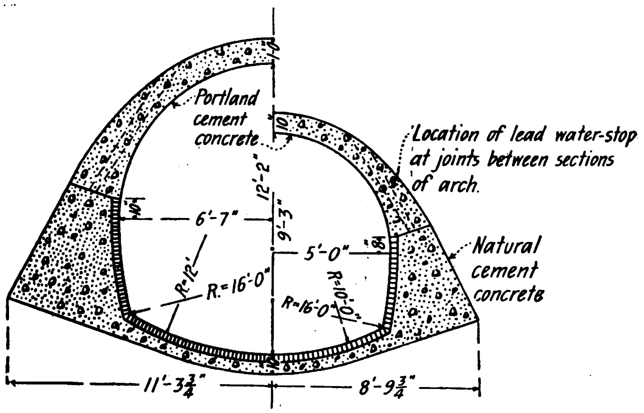


FIG. 197. — Weston Aqueduct.

which covers 65 acres. This basin, which has an average depth of 12 ft., continues the aqueduct for 4000 ft. to a screen-chamber, from which the aqueduct is built as a masonry conduit, for about 5700 ft., to a terminal chamber from which the water is distributed in the Metropolitan District by means of cast iron pipes.

There are five tunnels, aggregating 12,165 ft. in length, on the line of the aqueduct. They are lined throughout with concrete. Two valleys are crossed by pipe siphons, 3605 and 1123 ft. in length respectively. Each siphon is to consist of 3 riveted steel pipes, 7½ ft. in diameter. Only one line of pipes was laid in each siphon as the first installment. At the crossing of the Sudbury River, the siphon-pipe is constructed as an arch bridge of 80 ft. span, having a rise of 5.5 ft. The steel pipes are connected at each end with masonry gate-chambers by cast iron reducers,

* *Eng. Record*, May 4, 1901, p. 418; Oct. 18, 1902, p. 362.

built in the masonry. The inlet opening of each reducer is $9\frac{1}{2}$ ft. high by $4\frac{2}{3}$ ft. wide, and the form of the casting gradually becomes circular and terminates in a bell into which the steel pipe is leaded.

Thirty-nine culverts were built, to carry water courses across the line of the aqueduct. Nine culverts are carried as siphons under the aqueduct; three cross over the top of the conduit and 27 drain beneath the aqueduct. Many of the culverts consist of cast iron pipes, 12 ins. or more in diameter, having masonry head-walls at each end. At thirteen of the culverts which drain freely under the aqueduct to suitable water courses, 8-in. blow-off valves are set in small depressions in the bottom of the aqueduct for emptying it. Manholes with granite copings and steel covers are built at each blow-off and, also, at intermediate points, so as to make the blow-offs about $\frac{1}{4}$ mile apart.

304. The aqueduct of Los Angeles, Cal.,* was constructed in 1907–1913, to obtain a supply of water from the drainage area of Owens River, in the Sierra Nevada. This water-shed, which is about 120 miles long and 6 to 10 miles wide, contains about 2800 square miles, above the intake of the aqueduct. The rainfall at the floor of the Owens Valley region averages only about 5 ins. per annum, but increases with elevation to 30 and even 40 ins. at the crest of the Sierra Range. In the mountains, the precipitation is mainly snow. Owing to these circumstances, the flow in the river is very regular, and floods occur rarely. The melting of the snow in the mountains causes high water in the river from about the middle of May to July. In order to control the riparian rights in the region from which the water was to be taken, the city bought about 125,000 acres of land in Owens Valley from the United States Government and additional land from private owners. This land contains much ground water, which can be pumped into the aqueduct when it becomes necessary.

Two large reservoirs are to be constructed in Owens Valley, to impound water, and to equalize the differences in the annual flow of the river, namely: Long Valley reservoir with a capacity of 111,110,000,000 gals., and Tinemaha reservoir which will store 41,486,000,000 gals. These reservoirs have not yet been built.

305. *The Aqueduct.* — The intake of the aqueduct, located about 12 miles north of the town of Independence, consists of a concrete diversion dam and weir, 325 ft. long, provided with four radial 7.5- by 8-ft. gates. The whole flow of the river is diverted here into the aqueduct.

For the first 21 miles, the aqueduct is an unlined earth canal, having an average depth of 10 ft. (Fig. 198). This section has a capacity of

* "Final Report on the Construction of the Los Angeles Aqueduct," Los Angeles, Cal., 1916. Figs. 198 to 203 are taken from this report.

800 cu. ft. per second. As the ground water on this stretch of the aqueduct is only about 2 to 4 ft. below the surface, it enters the canal freely and adds to its supply. For the next 40 miles, the aqueduct is built along the eastern slope of the Sierra Nevada as an open canal, 12 ft. deep, with slopes of 1 to 1 (Fig. 199). The bottom and the slopes of the canal are covered with 6 inches of concrete, laid without forms. As

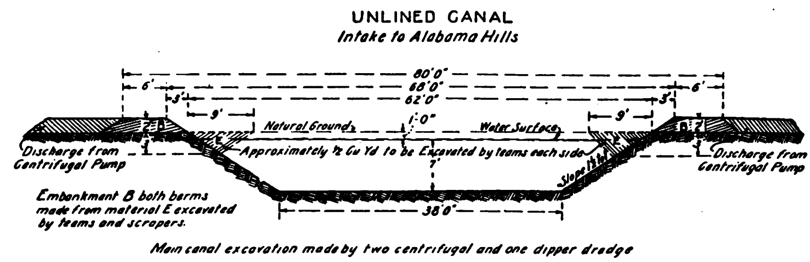


FIG. 198. — Unlined Canal of Los Angeles Aqueduct.

many small tributaries of Owens River flow into the canal, the capacity of this stretch of aqueduct is increased to 900 cu. ft. per second.

At the end of this section, the aqueduct discharges into the Haiwee reservoir, a regulating basin of 20,788,000,000 gals. capacity, which was constructed in what was the ancient bed of Owens River. The reservoir, which is 7½ miles long and ½ to 1¼ miles wide, was formed by

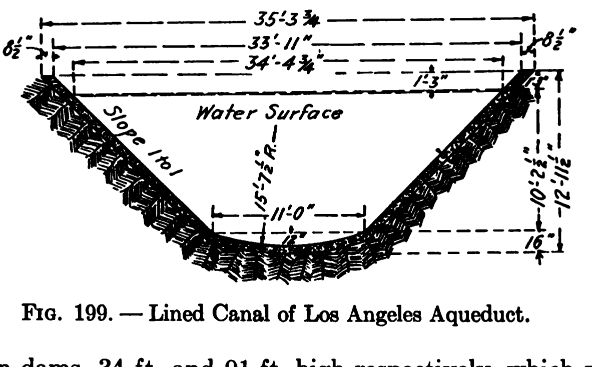


FIG. 199. — Lined Canal of Los Angeles Aqueduct.

two earthen dams, 34 ft. and 91 ft. high respectively, which were built by the hydraulic process across the old river-bed. The outlet from the reservoir is a tunnel, 10 ft. in diameter and 1193 ft. long. The flow into this tunnel is controlled by a cylindrical gate-tower, built of concrete, 80 ft. high and 21 ft. in diameter on the outside. It is provided with five pairs of sluice-gates, placed at different elevations. Each pair of gates can discharge 950 cu. ft. of water per second. Just below the reservoir, there is a hydraulic drop of 190 ft., which is to be used for future hydro-electric development, by connecting a steel penstock pipe with the outlet tunnel.

For about 2 miles from the end of this tunnel, the aqueduct consists of an open concrete-lined ditch. It is then continued, for about 133.5 miles, as covered concrete conduits, concrete-lined tunnels, and steel siphon pipes to the Fairmont reservoir, into which the water is dropped — 86 ft., when the reservoir is empty — from the standard conduit section, through an open ditch, lined with rubble concrete and having slopes of 1½ to 1. The reservoir, which has storage capacity of 2,483,000,000 gals., was formed by constructing, by hydraulic sluicing, an earthen dam, 1516 ft. long and 115 ft. high, across a natural basin. The main

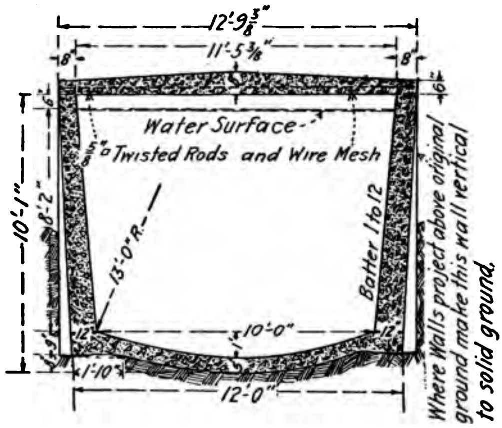


Fig. 200. — Covered Conduit of Los Angeles Aqueduct.

function of this reservoir is to serve as a forebay to two large power plants, built, about 8 miles below the reservoir, on the line of the aqueduct, and requiring 1000 cu. ft. per second for peak loads.

The capacity of the aqueduct between the Haiwee and Fairmont reservoirs was made 430 cu. ft. per second, 30 cu. ft. per second being allowed for possible losses by evaporation, leakage, etc. The covered conduit (Fig. 200), which is slightly varied in different sections of the work, has its sides and bottom lined with 6 ins. of concrete. The cover is made of reinforced concrete, generally 6 ins. thick at the sides and 8 ins. thick at the center. The tunnels on this stretch of the aqueduct were driven through material varying in hardness from granite to soft tufa. Their cross-sections vary, according to the grades adopted. The tunnels are 7.5 to 8 ft. wide, have an average height of 9 ft. from the invert to the crown, and are lined with concrete, 6 to 14 ins. thick (Fig. 201). Broad valleys or steep canyons are crossed by siphons of steel or concrete (Fig. 75, p. 253).

From the Fairmont Reservoir the aqueduct is continued by a pressure tunnel, about 5 miles long, driven through the Sierra Madre mountains (Fig. 202). This tunnel is 10 ft. 10 ins. high, 9½ ft. wide, and has a capacity of 1000 cu. ft. per second. It is continued by an adit, 2.6 miles long, to a surge-chamber at the head of the penstocks leading to Power-house No. 1, in the San Francisquito canyon. The outlet from the Fairmont reservoir is controlled in a circular concrete outlet-tower, having an inner diameter of 18 ft. Its walls, which are 5 ft. thick at the bottom, and 3 ft. thick at the top, are heavily reinforced with steel, both

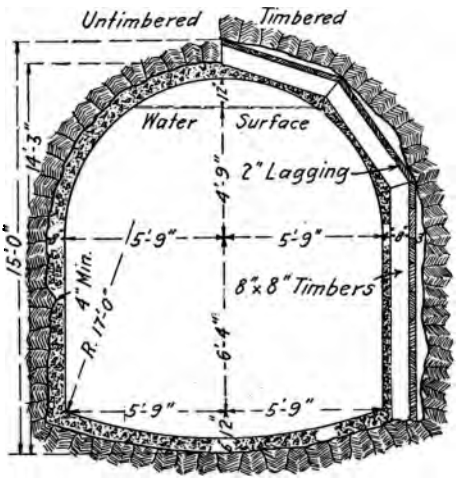


Fig. 201. — Standard Tunnel of Los Angeles Aqueduct.

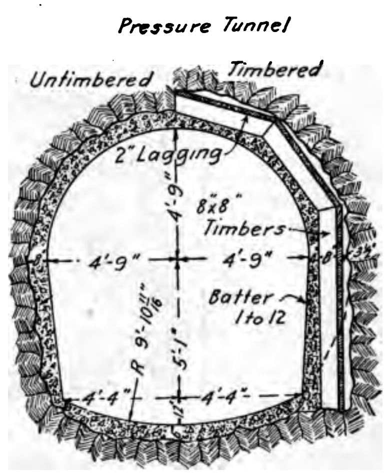


Fig. 202. — Elisabeth Pressure-tunnel, Los Angeles Aqueduct.

in a vertical and horizontal direction. Four tiers, of three gates each, are placed in the tower. Each tier can discharge the full quantity of 1000 cu. ft. per second.

From the surge-chamber to the power site in San Francisquito Valley, there is a power drop of about 946 ft. Below this site, the aqueduct is continued as pressure-tunnels or siphon pipes to a second power station in the same valley, where there is a power drop of 530 ft.

Below the lower power site in the San Francisquito Valley, the water is conveyed by tunnels having a capacity of 1000 cu. ft. per second to the Dry Canyon reservoir, which was constructed to regulate the fluctuating flow through the power plants, and to bring it back to the normal delivery of 400 cu. ft. per second. The water enters the reservoir through a rubble-lined open conduit. The reservoir, which has a capacity of 431,713,000 gals., was formed by an earthen dam, 528 ft. long with a maximum height of 61 ft.

The water level of the reservoir fluctuates about 5 ft. In order to obtain a constant discharge of 400 cu. ft. per second for the water supply of the city, a floating weir, consisting of an annular sheet iron tank, 30 ft. in inner diameter, 4 ft. thick and 12 ft. high, fitting around a concrete wall, was built. This floating tank slides up and down on the concrete wall, as the water level of the reservoir varies. Guide bearings are provided on columns, to prevent the tank from binding. Stop-cocks are placed in the tank, to make it possible to sink it to the depth required to enable the weirs to discharge 400 cu. ft. per second. When the float sinks too deep, water must be pumped out of the tank to bring it to the proper level. The water flowing over the tank falls into the basin inside of the concrete wall, from which it is taken, by a short inverted siphon, into an open forebay at the portal of the tunnel. Emergency gates are provided in the walls of the forebay to make it possible to draw water when the floating gate is out of order.

From the Dry Canyon reservoir, the aqueduct is constructed for about 11.5 miles as covered conduits, lined tunnels, and siphons to its terminus on the north side of the San Fernando Valley. Here the water drops 160 feet in a cascade, and flows for about 1.6 miles in an open concrete-lined ditch to the inlets of the two San Fernando reservoirs. The upper one of these reservoirs, which is to store about 4,887,400,000 gals., has not yet been constructed. The water is discharged, for the present, directly into the lower reservoir, which has a storage capacity of 7,494,000,000 gals. with its flow line at an elevation of 1135 ft. above the sea. This reservoir, which was formed by the construction of a high earthen dam, is so arranged as to draw the water from the by-pass conduit directly into the San Fernando siphon, or from the reservoir into the siphon. This reservoir acts as a storage reserve for the city's domestic supply, as well as for irrigation in the San Fernando Valley. From the Cascades to the Lower San Fernando reservoir, there is a fall of about 326 ft., which is to be utilized for generating electricity.

From the Upper San Fernando reservoir, the water is to be conveyed for about 12.8 miles through concrete-lined tunnel and riveted steel pipes to the inlet of a distribution reservoir, known as the Upper Franklin reservoir. This basin stores 42,000,000 gals., with its flow-line at 850 ft. above sea-level. From this reservoir the water is discharged through about 1.1 miles of riveted steel pipes into the Lower Franklin reservoir, which has a storage capacity of 360,000,000 gals., with the flow-line at elevation 575 above sea-level. A riveted steel pipe, about

7 miles long, connects this reservoir with the city's distributing pipes.

306. *Hydroelectric Power Developments.* — During the construction of the aqueduct and reservoirs, electric power for operating part of the machinery was obtained from power developments on the Cottonwood and Division Creeks, tributaries of the Owen River. These power developments, the four power drops along the line of the aqueduct, described above, and others to be acquired by the city, will furnish about 200,000 horsepower to meet peak load demands, and will give the city cheap electricity in addition to the large supply of pure water secured by the works described above.

307. *Engineers.* — The preliminary plans for the aqueduct and reservoirs were made under the direction of the Water Commissioners of Los Angeles by William Mulholland, chief engineer, and the final plans adopted were approved by a Board of consulting engineers, consisting of Frederick P. Stearns, John R. Freeman, and James D. Schuyler. The construction of the works was entrusted to the Public Service Commissioners of said city who appointed William Mulholland as chief engineer and J. B. Lippincott as assistant chief engineer. These engineers remained in charge of the works until their completion.

308. The Rochester Pipe Conduits.* — The water works of Rochester were constructed in 1872–1876. They differ from those of most American cities in having a dual system, the water for domestic purposes being obtained by gravity from Hemlock Lake, about 28 miles south of the city, while water for public and industrial purposes is pumped directly from the Genesee River, which flows through the city. Hemlock Lake has an area of 1828 acres, at low-water, and is supplied from a drainage area of 43 square miles. It lies 286 ft. above the general city level. The original pipe-line conveying the water from this lake to Rochester consists of about $9\frac{3}{4}$ miles of 36-in. wrought iron pipe and of $18\frac{1}{2}$ miles of 24-in. pipe of which nearly 3 miles are wrought iron pipe, in two sections, the remaining part consisting of cast iron hub-and-spigot pipe. The wrought iron part of this conduit has proved to be very satisfactory. Considerable trouble has been experienced, however, with the cast iron pipes from tuberculation and, occasionally, from leakage at the joints.

309. *New Conduits.* — The city having outgrown its waterworks, it was decided in 1890 to construct a new conduit from Hemlock Lake to the city. The new works were designed and built under the direction of Emil Kuichling, chief engineer.

The new conduit, which was completed in 1894, consists of a 60-in. steel intake-pipe, about 1500 ft. long, laid in the lake from a submerged crib

* *Eng. News*, April 11, 1895, p. 234; *Eng. Record*, April 13, 1895, p. 346, *et seq.*

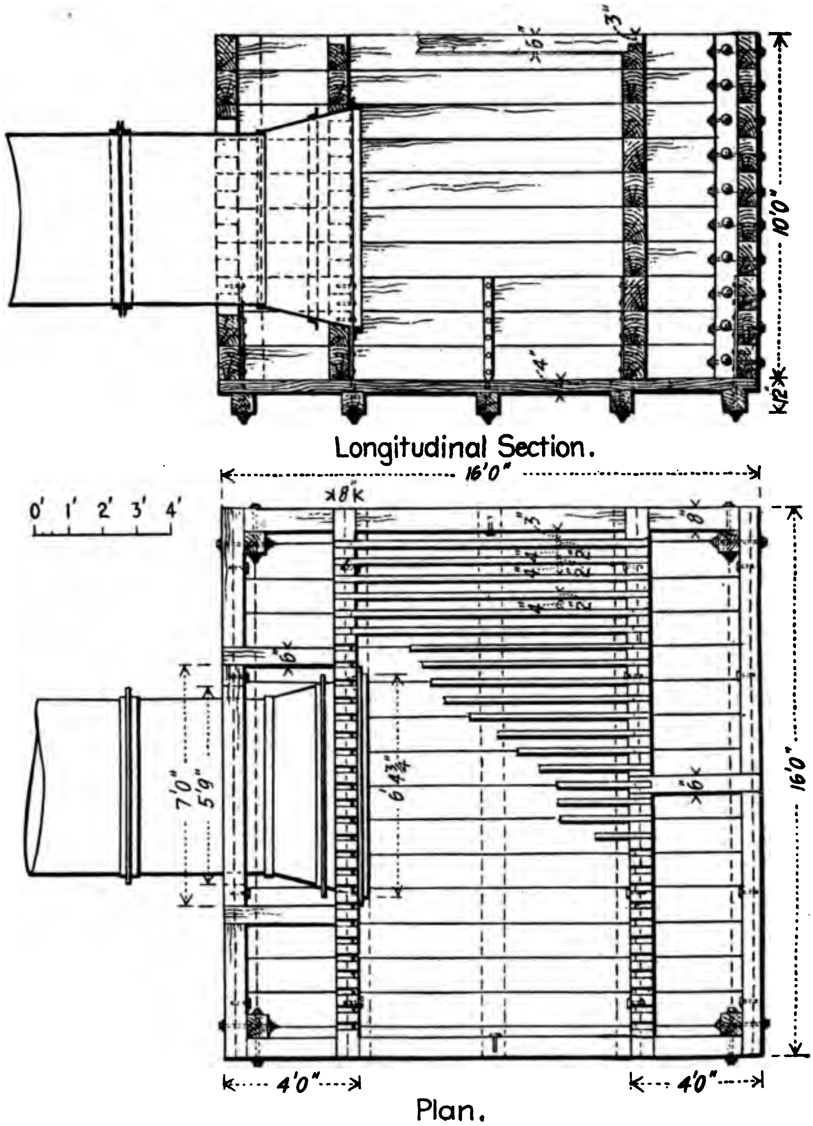


Fig. 203. — Intake Crib of Rochester, N. Y.
(Built in 1893 and 1894.)

to the shore, then of 12,000 ft. of brick conduit, built partly in tunnel, and lastly of 26½ miles of 38-in. riveted steel pipe.

Fig. 203 shows the submerged crib, forming the inlet to the conduit. The water is drawn about 26 ft. below the surface of the lake. The 60-in. intake pipe is described on p. 226. It was shipped from the shops in Pittsburgh, Pa.,* in 30-ft. sections. Three of these sections and a ball-and-socket joint were riveted together on shore so as to form a length of about 100 ft. of pipe, weighing 14 to 16 tons. These sections of pipes were floated into position, and sunk into a trench which had been dredged for them in the lake bottom, that consists of clay at this place.

A masonry gate-house for controlling the flow from the lake was constructed at the point where the steel intake-pipe joins the brick conduit. It is provided with a double set of passages to the conduit, one of which can be shut off for inspection or repairs, while the other maintains the flow to the city. An overflow chamber is provided at the junction of the masonry conduit with the 38-in. steel pipe. Another overflow is constructed midway in the long stretch of 17½ miles, from the beginning of the 38-in. pipe to Rush reservoir. It is formed by a short stand-pipe, rising from the conduit on top of a hill to the hydraulic grade-line. This arrangement limits the pressure to which the steel pipe can be subjected when the gate at its lower end is closed.

310. *The Specifications* required the steel pipe to be made of soft, open-hearth steel, containing not over 0.6 per cent of manganese and 0.06 per cent of phosphorus. It was to have a tensile strength of 55,000–65,000 lbs. per sq. in., with an elastic limit of not less than 30,000 lbs. and an elongation of 22½ per cent in 8 ins. Cold-bending, punching and other tests were required. All plates having at any point less than 95 per cent of the required thickness were rejected.

311. *The pipe* forms a continuous riveted tube, without any provision for expansion, as it was calculated that the internal stresses due to the changes of temperature to which the pipe might be exposed would be far within the elastic limit of the material. Under these circumstances, it was essential that the strength of the net section of the plate at the circular seams should be equal to the shearing resistance of the rivets therein. The joints were designed to meet this condition. Motion was prevented, by bedding the pipe at the ends of the conduit in large masses of masonry. Similar precautions were taken in the design of all stop-valves and branches which were inserted in the pipe-line. These appurtenances were provided with flanges, and were made much stronger than the pipe, to which they were connected by strong bolts.

The pipe was made in lengths of about 7 ft., having alternately an inner diameter of 38 ins. and of 38 ins. plus twice the thickness of the plate. The smaller sections fitted, therefore, just inside of the larger sections. Each course or section of pipe was made of a single sheet.

* The pipe was made by the Carroll-Porter Boiler and Tank Co., Pittsburgh, Pa.

All straight seams were required to be made in the upper quarter of the circumference of the pipe. Four different classes of pipe were used, according to the pressure that was to be resisted, as given in the following table:

TABLE XLIV. — ROCHESTER 38-INCH STEEL PIPE

Static head	Thickness of steel plate	Longitudinal joint
	Inch	
Less than 120 ft.	$\frac{1}{4}$	Single riveted
120–153	$\frac{1}{4}$	Double riveted
153–199	$\frac{5}{16}$	Double riveted
199–263	$\frac{3}{8}$	Double riveted

The circumferential joints were always made single-riveted, except where two different kinds of pipe were joined, in which cases the seams of the thinner pipe, including the junction seam, were double riveted for a distance of about 200 ft. This was done, in order to make the circumferential seams of the thinner pipe approximately equal to those of the thicker pipe. It was calculated that in a distance of about 200 ft. the friction of the earth against the pipe, after the trench had been properly refilled, would balance the resultant of the longitudinal forces, produced in the two classes of pipe by the extreme variation of temperature in the conduit, which was taken as 45° Fahrenheit. Each class of pipe is regarded as firmly anchored in the soil, except for the above-mentioned distance at the junctions.

Four courses of pipe, making lengths of about 27$\frac{1}{3}$ ft., were riveted together in the shop and thoroughly caulked on the inside and outside. Changes in direction in either the alignment or grade were made by slightly beveling at one end a sufficient number of courses. The contractors were supplied with accurate drawings on a large scale, showing these changes. In order to avoid having many patterns, a standard bevel of 2° 43.5′, corresponding to an offset of 1.806 ins. in the diameter of 38 ins., was used wherever possible. The standard bevel on the end of every fourth course forms a polygon fitting a 10° curve very closely. If applied at the end of every second course, the polygon would fit a 20° 8′ curve, and if placed at the end of every course, it would fit a 41° 9.75′ curve. The radii for the three courses mentioned above are, respectively, 573.7, 286.1, and 142.2 ft.

312. *Coating.* — Great pains were taken to obtain the best coating for protecting the pipes from corrosion. In the beginning a purely asphaltic coating, like that used successfully on numerous conduits in California, and, also, on the Newark pipe (p. 334) was applied. The pipes, after being thoroughly cleaned, were heated in an oven to about

320° Fahrenheit, and were then dipped into a large tank containing the coating mixture, in which they were allowed to remain until the metal of the pipe acquired the same temperature as the bath. The pipes were then removed, the surplus coating was allowed to drip off, and the pipes were then placed on skids, where the material was left to harden. This coating, which appeared to be very satisfactory, when exposed to water, was found to undergo a change, if the pipes were exposed to the air for several weeks. It failed, also, to adhere tightly to the metal. The contractors were put to great expense in painting with costly materials large areas from which the coating had scaled off. California asphalt and, also, a mixture of Trinidad asphalt and the best grade of coal-tar were used, the latter giving the better results.

313. *Japanning Process.* — Prof. A. H. Sabin, the chemist of the well-known firm of Edward Smith Co. of New York, varnish manufacturers, was later engaged to make a series of experiments to find a perfectly satisfactory coating. He finally devised a very promising japanning process, which was used for the remaining pipes. The essential point, in which this process differs from the usual method of coating, is in removing the pipes from the bath of asphalt and tar to a large brick oven, in which they are baked at a high temperature for about ten hours.

At Rochester each pipe was heated in an oven, large enough to contain three pipes at a time, before it was dipped in the bath of coating mixture, which was placed in a vat forming half of a horizontal cylinder, 6 ft. in diameter and 32 ft. long. The vat had brick walls which were lined with iron. Crude oil was used as the fuel in the furnace that was placed directly under the vat. The baking oven was about 9×25 ft. in the clear, and sufficiently high to admit a twenty-eight foot section of pipe, placed on end. Twelve of these sections could be baked at a time. A temperature of 500–600° Fahrenheit was maintained in the oven.

Great care was taken to protect the coating from injury during the transportation and handling of the pipes. All chains, bolsters or saddles, on which the pipes bore, were covered with old rubber hose, canvas or carpet. During the latter part of the work, canvas coverings, or mats, were placed on top, or within the pipe, wherever the workmen were engaged, and all who entered the pipes were required to put on soft felt shoes or overshoes. Some abrasions of the coating in fitting the pipes together and from the riveting were unavoidable. The coating was replaced in such cases by "P. & B." paint, which consists of refined asphalt dissolved in bisulphide of carbon. The latter material is a very volatile liquid which evaporates, leaving a thick coating of asphalt on the metal. As the vapor rising from this paint, when first applied, is very offensive in odor and, also, inflammable, the pipes had to be well ventilated while the paint was being applied. This was accomplished by opening the manholes which had been provided on top of the pipe, at intervals of about 1000 ft., to permit ingress to the pipes for inspection or repairs, and by forcing a strong current of air through the pipe, by means of a suitable fan or blower.

314. *Pipe Laying.* — In laying the pipe, two sections were riveted together making a length of about 54.5 ft., before they were lowered into the trench. These long sections, which weighed 7000–10,500 lbs., were rolled by hand on top of timbers placed over the trench, and were then lowered by means of one or two strong derricks.

When about a mile of conduit had been completed, it was tested by water pressure. The ends of the pipe were closed, either by temporary steel boiler-heads, or by gate-valves, and the pipe was then filled with water, by means of a force-pump. The water-pressure was raised gradually to a pressure of 50 per cent greater than that to which the pipe was to be subjected.

315. *An air-valve* of special construction * was placed at each summit of the conduit line. It serves to let the air out of the pipe, when it is being filled with water, and to admit air automatically when the water is drawn out of the pipe, either by the blow-offs, or on account of a break. The air-valves are simply bronze poppet-valves 3, 4 or 6 ins. diameter, which are either used singly or in groups of two or four. They are kept closed by springs, and are attached to the conduit in the manner shown in Fig. 204. The air-valves are protected from frost and injury by an iron-casing, covered with earth, from which a standpipe rises 3 ft. above the surface. It is covered with an iron hood having a small opening (usually kept covered with a locked flap) through which a key can be pushed to open one of the valves during the filling of the pipe. As the springs are made only strong enough to balance the weight of the valve, the valves open automatically, whenever the pressure in the pipe is reduced below that of the atmosphere.

316. *Blow-offs* for emptying the pipe for inspection or repairs are provided at every depression. They consist of short branch pipes, 6 ins. and, in one case, 8 ins. in diameter, controlled by two gate-valves, the one nearer the conduit being only shut when the other one is out of order. The blow-off pipes are connected to the conduit by means of a flanged casting, riveted to its lower quarter. At each blow-off, the main conduit and the two gate-valves of the blow-off are supported on a foundation of concrete, in order to avoid breakage by unequal settling, etc.

* Invented by Emil Kuichling, M. Am. Soc. C. E., and manufactured by the Rensselaer Valve Co., Troy, N. Y.

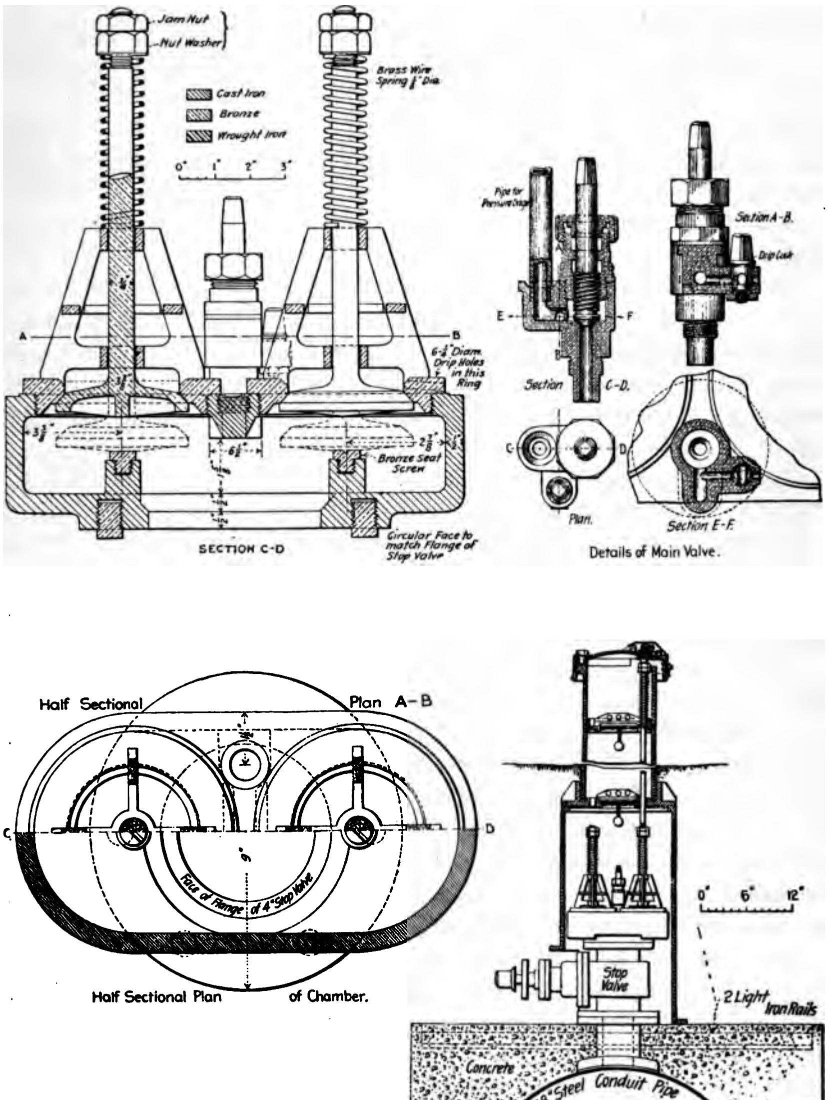


FIG. 204. — Cluster of Air-valves on Rochester Pipe-line.

About 110,000 ft. from the inlet, the conduit is connected by means of a 36-in. cast iron branch pipe, about 1000 ft. long, with Rush reservoir, which is about forty feet above the main conduit. Forty thousand feet further (*i.e.*, 150,000 ft. from the inlet), the conduit terminates at the Mt. Hope reservoir in the city of Rochester.

317. *Gate-valves.* — Heavy 36-inch gate-valves are placed in the pipe-line at different points. Ten are set between the inlet and the branch to Rush reservoir, and six between the latter point and Mt. Hope reservoir. They serve for shutting off the section between any two gate-valves for inspection or repairs.

318. *Pipe Connections.* — The old and new conduits are connected by means of the branch pipe from the latter to Rush reservoir, and, also, by means of the Mt. Hope reservoir. Two 36-in. mains distribute the water from the latter reservoir. The surface of Rush reservoir is about 125 ft. higher than that of Mt. Hope reservoir. Arrangements are made for turning the full pressure from Rush reservoir into the two 36-in. distributing mains, which in this case are closed from Mt. Hope reservoir by means of check-valves.

319. The first Newark conduit, was constructed in 1889–1892 by the East Jersey Water Co., which entered into a contract with the city of Newark to furnish a new supply of water for the city by gravity from the Pequannock, Wynockie or Ramapo, three tributaries of the Passaic River. According to the agreement, the works were to have a capacity of 50,000,000 U. S. gals. per day, of which quantity, one-half was to be delivered for low-service and the other half for high-service, with a head of 300 ft. above mean tide. The works were to be completed by May 1, 1891, only 2 years and 7 months being allowed for their construction.

The works were designed by Clemens Herschel, M. Am. Soc. C. E., the chief engineer of the East Jersey Water Co., and were executed under his direction.* The supply was obtained entirely from the Pequannock. Two reservoirs were constructed in the water-shed of this stream, to store a supply for 120 days. The water flows from these basins in the natural channel of the stream to a small intake reservoir, constructed on the Pequannock (known as the Macopin intake) from which a 48-in. riveted steel pipe conveys the water to Newark. The capacity, elevation, etc., of each of these reservoirs is given in the following table:

* Paper by Clemens Herschel, M. Am. Soc. C. E., in Jour. New England Water Works Assoc., 1893, Vol. VIII, pp. 18–42; see also, *Eng. Record*, Aug. 8, 1891, p. 156; *Eng. News*, Oct. 11, 1890, Aug. 1 et seq. 1891 and June 15 et seq. 1893.

TABLE XLV. — STORAGE RESERVOIRS FOR THE WATER SUPPLY OF NEWARK

Name of reservoir	Drainage, area square miles	Surface, acres	Elev. of spillway above mean tide	Maximum depth of water	Cubic contents, U. S. gallons
Clinton	9.50	423	992.0	41.3	3,518,000,000
Oak Ridge	27.30	383	836.0	44.5	2,555,000,000
Macopin intake	25.90	12	583.7	14.4	32,000,000
	62.7	718			6,105,000,000

320. *Steel Conduit.* — The great pressure to which the conduit conveying the water was to be subjected, and the rapidity with which it had to be built, led to the construction of a continuous riveted steel conduit. As this pipe was the first of its kind, of any extent, in the Eastern States and as Mr. Herschel introduced several new features in its construction, we shall describe it somewhat in detail.

The main conduit has an inner diameter of 4 ft. and a length of about 21 miles, from the inlet at Macopin reservoir to the low-service receiving reservoir at Belleville.* It has a hydraulic grade of 2 in 1000. A branch conduit, 36 ins. in diameter and five miles long, constructed like the main conduit, conveys part of the water from a point near the Belleville reservoir to a high-service reservoir at East Orange Avenue, Newark.

The flow through the conduit is controlled entirely by a sluice-gate, placed at its up-stream end. By this arrangement, the pipe is only subjected to the pressure due to its hydraulic grade, and has never to bear the full hydraulic head from the level of the inlet reservoir. This simple plan, which appears to have been introduced practically for the first time in the Newark conduit, effected a saving of fully 40 per cent in amount of metal required in the pipe. A telephone line, constructed along the conduit, gives an easy means of sending the necessary order to the gate-keeper at the inlet. If by any accident the inlet-gate should not be closed, when ordered, the only result would be a waste of water at the overflow-weirs of the receiving reservoir, which are always kept open.

One of the most striking features of the Newark conduit is the total absence of any mechanical contrivances to allow for contraction or expansion. Riveted pipes have been built successfully in this manner

* The bottom of the conduit at the intake is at elevation 569.3 above mean tide. The hydraulic grade begins at the intake at elevation 579.2, reaches the low-service reservoir at Belleville at elevation 358, and the high-service reservoir at East Orange Ave., at elevation 300. The high-water mark of these two receiving reservoirs are respectively at elevation 168.4 and 226.6. It is intended to construct eventually a special high-service reservoir at elevation 300.

in California, but there was, at that time, no precedent for this method of construction in the Eastern States. In the wrought iron pipe, constructed in 1872–1876, to supply Rochester, N. Y. (page 324), hub-and-spigot joints were provided, at intervals of fifty-six to eighty-one feet, to allow for expansion and contraction. The experience made with the Newark pipe has shown that a riveted wrought iron or steel pipe may be made continuous, providing it is made strong enough to resist the strains caused by changes of temperature. In the Newark pipe, the temperature may vary from 32–77° Fahrenheit, a difference of 45 degrees. The resulting strains are resisted jointly by the friction between the pipe and the earth in which it is bedded, and by the elasticity of the material, the one causing the action of the other.

321. *Pipe-making.** — The pipe was made of the best open hearth steel, which was required to conform to the following specification:

"Tensile specimen to be eight (8) ins. long and one and one-half (1½) ins. wide, between measuring points. Tensile strength to be between the limits of 55,000 and 67,000 lbs. per sq. in. Elastic limit to be not less than 30,000 lbs. per sq. in. For plates ⅜ in. thick and heavier, elongation to be not less than 25 per cent longitudinally on the plate, 22 per cent transversely on the plate. Plates thinner than ⅜ ins., elongation to be not less than 22½ per cent longitudinally, and 20 per cent transversely on the plate.

"Bending specimen to be six (6) ins. long and one (1) in. wide, and to be hammered down flat, when cold, without showing cracks.

"Test to be made from 20 per cent of the plates selected at random.

"Steel not to contain more than six one-hundredths (0.06) per cent phosphorus, and six one-hundredths (0.06) per cent sulphur, and sixty one-hundredths (0.60) per cent manganese."

The high quality of the steel specified permitted the plates to be punched without any injury. About $85,000 might have been saved in the cost of the plates by using Bessemer steel, but in that case it would have been necessary either to drill the rivet holes or to punch and ream them.

The rivets used in the shops were made of soft open hearth steel, having a tensile strength of 57,000 to 63,000 lbs. per sq. in. and an elastic limit of 30,000 lbs. They were driven by hydraulic riveters. Those used in the field were driven by hand and were made of iron

* The steel plates were manufactured by Carnegie, Phipps & Co., of Pittsburgh at their Homestead Works. The pipe was made and delivered along the trench by McKee & Wilson, boiler makers, then of South Bethlehem, Pa. T. A. & R. G. Gillespie, of Pittsburgh, Pa., excavated the trench, placed the pipe, joined the sections and refilled the trench.

having a tensile strength of 55,000 lbs. per sq. in. and an elastic limit of at least 27,500 lbs.

The steel plates were shipped from Homestead, Pa., where they were manufactured, in new coal-cars, having deep bodies and removable flat roofs, to Paterson, New Jersey, where extensive shops were erected and equipped for making the pipes. The plates were all 7 ft. wide, about 13 ft. long, and $\frac{1}{4}$, $\frac{5}{16}$ or $\frac{3}{8}$ in. thick, according to the pressure to be resisted. Four plates (two large and two small sheets) were used to make a section of pipe about 27 ft. long. Two of the rings forming the section were made just large enough to slip over the other two rings, so as to make telescopic joints. The process of making the pipes was as follows: The rivet holes were marked and punched; the edges of the plates were beveled by a planer; the plates were bent to the true circle in rollers; and then fitted, riveted, and caulked, the riveting being done by hydraulic machines and the caulking by pneumatic tools, using compressed air.

322. *Coating.* — Each section of pipe was given two coats of California asphalt. This material is found in Southern California in two forms, *viz.:* As rock asphalt, containing 40–60 per cent of sand; and as liquid asphalt, which is combined with a species of petroleum. When the first kind is used, it must first be melted to get rid of the combined sand, and then remelted in the coating vats, enough coal tar being added to give the mixture the proper consistency. Liquid asphalt must first be distilled until only the bitumen remains. It is then melted in the coating vat, where it is usually mixed with the products of the same wells, taken at a lesser stage of distillation, either to make it more fluid or more viscous.

Two iron vats or tanks were used for the coating, each being long enough to hold a 27-ft. section of pipe. The furnaces were placed beneath the vats, and the coating mixture was kept at a temperature of about 300° Fahrenheit. Each pipe was left for about twenty minutes in the bath, so as to acquire the required temperature. It was then removed, and dipped about fifteen minutes later in a second tank, to receive a thickening coating. It was left less time in the second bath than in the first, in order to avoid melting the first coat. The specifications required the coating, after the two dips, to weigh four pounds per square foot of surface.

Two kinds of pipe were made at the shops, as required, *viz.:* straight pipes, and pipes beveled at one end for use on curves. All changes of alignment or grade were made by one of four standard curves, *viz.:* those of $2\frac{1}{2}$ degrees, 5 degrees, $7\frac{1}{2}$ degrees or 10 degrees curvature. The vertical curves were all 10 degrees. The curves were made by laying one to four straight sheets to form a chord on the lengths constituting the desired curve. At each angle-point between the chords, and at the tangents, a pipe beveled at one end was placed. Four pipes were shipped on a car from the shops, two being placed above the others. The pipes were hauled to the trench by teams. Two horses were generally able to haul a twenty-seven foot pipe, but on steep grades four horses had to be used.

323. *Pipe Laying.* — The trench for the pipe was generally 5 by 5 ft. in section. Most of the excavation was in earth, but some rock had to be blasted, and in some cases sheet-piling was required. In light, shifting or water-bearing soil, the pipe was protected by a thick layer of concrete. The pipe was delivered along the trench in sections 27 ft. long. Two of these sections were riveted together, rolled on top of skids, placed over the trench, and then lowered by tripod derricks into the trench. The circumferential joint with the section last laid was made by driving the rivets on the upper half of the pipe from without, and the rivets of the lower half from within, the riveters in the latter case working on their knees and using short handled hammers. At every other joint, a $1\frac{1}{2}$-in. tapped hole was provided near the rivet line. The rivets driven from the outside were passed through this hole to the *holder-on* within the pipe, and the hole was finally closed with a cast iron plug screwed in from the outside into carefully tapped screw threads.

The back-filling was kept up to the pipe-laying, as nearly as practicable, and was carried up above the pipe so as to make a ridge, about 2 to 3 ft. high above the level of the ground.*

When the pipe was delivered and the trench was dry, about 1000 ft. of pipe could be laid per day. The best record was one mile laid in three days. The average amount of pipe laying was about 19,000 lineal feet per month, the maximum being 22,165.

Where the workmen were engaged, the coating was protected, both on the inside and outside, by canvas, carpet, etc. The workmen, upon entering the pipe, were required to wear rubber boots. Wherever the coating was injured, the pipe was painted with "P. & B." paint (p. 328).

After the pipe was completed, it was carefully inspected on the inside by men crawling through the pipe. A large number of small objects, which had been left behind, while the pipe was being laid, notwithstanding constant care, were removed, and the water was finally admitted on Dec. 30, 1891.

324. *Bridges.* — The conduit is carried across the Pompton River and the canal adjoining it by a wrought iron bridge having four 150-ft. spans. The Passaic River is crossed by a similar bridge of three spans

* It is now customary to test a pipe before covering it with earth.

of 150 ft. These bridges are formed of Pratt trusses, placed ten feet center to center. The pipe, which weighs about 1000 lbs. per lin. ft. when full of water, is suspended from the top-chord panel joints. A narrow foot-path is placed at the level of the bottom-chord. For spans of less than 10 ft., the pipe is strong enough to support itself. Small streams are crossed by stone culverts up to 20 ft. in span. At a number of places the pipe passes under streams or canals.

325. *Shut-off Gates.* — The main conduit is divided into ten sections by nine 48-in. shut-off gates,* which are placed at convenient points. The branch conduit has two similar 36-in. gates. Each of the gates has two 10-in. by-passes, one on each side, which are controlled by 10-in. gate-valves. At each of the shut-off gates, excepting the first two, a blow-off valve is provided, which is interlocked by means of bars with the gate in such a manner, that the gate cannot be closed until the blow-off is opened, and, vice versa, the blow-off is closed before the gate is opened. By this arrangement, which was introduced in this conduit for the first time, the pipe can never be subjected to a greater pressure than that due to the hydraulic grade. The first shut-off gate from the inlet did not require a blow-off, as the pipe at that point could readily withstand the full hydrostatic pressure from the inlet reservoir. At the second gate, which is placed at Pompton Notch, the pipe almost reaches the hydraulic gradient. No blow-off is needed in this case as an open overflow is constructed at this point.

The arrangement of blow-off valves, interlocked with the corresponding gates, is much more reliable than the use of any relief valves, held in place by springs or weights. The shut-off gates are made extra strong and are provided with flanges. Where they are placed, the pipe is anchored by surrounding it with a prism of masonry, about 5 ft. square, extending 25 ft. each way from the gate. This arrangement relieves the gates of any great strains from change of temperature.

326. *Blow-offs and Manholes.* — A blow-off is provided at every depression on the pipe-line. Manholes are constructed every 1000 ft. on top of the main conduit, and every 500 ft. on top of the branch conduit. They give access to the interior of the pipe for inspection or repairs.

327. *Air-valves* are placed at all summits on the pipe-line. They serve to let out the air, when the pipe is being filled, and to admit it when the water is drawn off, either intentionally or by a break. Each air-valve† consists of a cluster of 4, 6, or 10 brass cup-valves, being

* All the shut-off gates and blow-off valves were made by the Eddy Valve Co., Waterford, N. Y.

† The air-valves were made by the Coffin Valve Co., Boston, Mass., which, also, made the sluice-gates at the intake gate-house.

equivalent to 6-in., 8-in., or 10-in. connections. The cup-valves have an outer diameter of $3\frac{1}{16}$ ins., are turned bottom up, and are light enough to float in water. They move between brass guides. The water in filling the pipe closes the air-valves by forcing the cup-valves against their seats, the shock in striking the seats being reduced by the air within the cup-valves, which forms a cushion. As soon as the water is lowered in the pipe, the cup-valves drop automatically and admit air. At each of the clusters of air-valves, a special valve, worked by hand, is placed, to let out the air that may accumulate in the pipe. A gate-valve is provided for each cluster of air-valves.

The air-valves are proportioned to admit sufficient air to prevent the pressure in the pipe from being reduced below half an atmosphere, in case the pipe should be suddenly cut in two at its most dangerous point. They were temporarily and hastily protected from frost, injury, and interference by malicious persons by special boxes in the following manner: a cellar-pit was built around each air-valve and was covered with loose boards, on top of which a cheap form of quilt, made in halves, was laid. Over this was placed the box proper, which is pierced by two or four square wooden chimneys. To prevent mischievous persons from throwing in stones, sticks, etc., the chimneys are hooded.

328. *Pressure-regulators.* — Until the city of Newark completed its special high-service reservoir, relief-valves or pressure-regulators * were placed at the two terminal gate-houses at the reservoirs. These regulators are contrived in such a manner that a small relief-valve, which can be set, by means of a hand-wheel, at any desired pressure, opens when this pressure is reached and admits an excess of pressure into a chamber, which depresses the main valve, and thus reduces the pressure in the pipe. To insure safety, the piping and relief valve were made in duplicate. Three of these pressure-regulators, each governed by two independent relief-valves, were put in the Belleville gate-house.

329. *Venturi Water-meters.* — According to the agreement made between the city of Newark and the East Jersey Water Company, the city was to pay four million dollars for the water works on May 1, 1892, and an additional two million dollars on September 24, 1900, when the works were to become the property of the city. Until the last payment was made, the Water Company was to have the right to sell all the water supplied by the works, which Newark did not consume. Under these circumstances it was necessary to keep accurate records of the quantity of water drawn at different points from the conduit. The discharge was measured by means of Venturi water-meters (p. 530), two 48-in.,

* Made by the Ross Valve Co., Troy, N. Y.

one 36-in. and one 16-in. being the first used. By February, 1900, the East Jersey Water Company had installed fourteen of these meters, in sizes of 12 to 51 ins.

At the time when the conduit was designed, there were no reliable data of the discharge of a riveted pipe of the size contemplated. The only gauging of a pipe near the size of the Newark conduit were those made for the Rochester pipe, built in 1873–1876, which was partly 24 ins. and partly 36 ins. in diameter. These gaugings, which were published in the report of the chief engineer of the water works, of January 1, 1877, have since been shown to have given discharges that were much too large. As the determination of the diameter of the Newark pipe was based on the Rochester gaugings, for lack of any other data, it was found upon testing that the Newark conduit discharged only about 36 million gallons per day, instead of the 50 millions required by the contract. This deficiency in capacity was remedied by building a second conduit, parallel with the first pipe which was made 48 ins. in diameter at its upper end, where the conditions of grade were less favorable than on the lower part of the line, and 42 ins. in diameter the rest of the way.*

330. The second Newark conduit was constructed in 1895 to 1897 from the Macopin Inlet to Belleville, a distance of 5½ miles. It was connected temporarily at the latter point with the first conduit, increasing the discharge of this conduit below this junction to 45.6 million gallons per day.

331. The Coolgardie Pipe-line.† — The town of Coolgardie is situated in Western Australia, about 350 miles from the west coast and 250 miles from the south coast, in a district noted for its dryness, in which the annual rainfall has been as low as 3½ ins. Owing to the great porosity of the soil, the rain water is soon absorbed, and becomes saline from the salts the soil contains. The discovery in 1892 of great goldfields near Coolgardie brought many miners and settlers to this district. As there were no available sources of water supply in the vicinity, the Government built expensive works in 1898 to 1903, to obtain water from the Darling Ranges on the western coast to Coolgardie, a distance of over 300 miles.

By means of a masonry dam, a storage reservoir of 4,600,000,000 gals.‡ capacity, fed by a watershed of 569 miles, was formed on the Helena River at Mundaning. It stores a sufficient supply for two years.

* See "115 Experiments on the Carrying Capacity of Large Riveted Metal Conduits, up to Six Feet per Second of Velocity of Flow," by Clemens Herschel, M. Am. Soc. C. E., New York, 1897.

† Proc. Inst. C. E., 1905, CLXII, p. 50.

‡ Imperial gallons are used in this description.

From this reservoir, the water is conveyed by a line of 30-in. steel pipe, 308 miles long, to a distributing reservoir of 12,000,000 gals. capacity at Bulla Bulling, from which the water flows by gravity, through a continuation of the pipe-line, 21 miles to Coolgardie and 23½ miles farther to Kalgoorlie, the total length of the steel conduit being 351½ miles. The water is raised in this distance a total height of 1290 ft., by means of eight pumping stations, located at different points along the pipe-line.

The first pumping station is placed on the Helena River, 650 ft. downstream from the storage reservoir. The pumps draw their water from a 4-ft. stand-pipe, erected in front of them, and lift the water a net height of 415 ft. through 1½ miles of pipe to a concrete receiving tank of 468,000 gals. capacity. The pumps of Station No. 2 draw their water from the first receiving tank, and raise it a net lift of 340 ft. into a regulating tank at Baker's Hill, having a capacity of 500,000 gallons. From here the water flows by gravity a distance of 12 miles to a regulating tank at West Northam, from which the water flows a further distance of 41 miles to the Cunderlin reservoir, which has a capacity of 10,000,000 gals. Pumping Station No. 3 is located about ¾ mile from this reservoir, and the pumps draw their supply from a stand-pipe, as at Station No. 1. The section between pumping stations Nos. 3 and 4 is 62¾ miles long, and the net lift is 215 ft. The water is delivered at Station No. 4 into a concrete tank, having a capacity of 1,000,000 gals. From Station No. 4 the water is raised a net height of 333 ft. through a pipe 32½ miles long, into a rectangular concrete receiving-tank, storing 1,000,000 gals.

The arrangements at Stations Nos. 5, 6, 7, and 8 are similar to those at Station No. 4, and the receiving tanks Nos. 6, 7, and 8 are similar in design to that of No. 5, and have the same capacity. The net lifts at Stations Nos. 5, 6, 7, and 8 are respectively 52, 106, 56, and 183 ft., and the corresponding lengths of sections are 46, 31¾, 45, and 12½ miles. From Station No. 8 the water is delivered into the Bulla Bulling distributing reservoir, from which the water flows by gravity to Coolgardie and Kalgoorlie. At each of these towns a circular service-reservoir is constructed, their capacities being respectively one and two million gallons.

332. *The Pipes.* — Cast iron pipes could not be used, on account of the cost and difficulty of the freight, both by sea and by land. Bids were received for three different kinds of steel pipes for the pipe-line, *viz.:* riveted pipes, welded pipes, and lock-bar pipes. According to the lowest tenders, lock-bar pipe was found to be 50 per cent cheaper than welded pipe, but to cost 11½ per cent more than riveted pipe. The latter difference was considered to be more than offset by the advantage lock-bar pipe possesses over riveted pipe, *viz.:* greater strength, greater water-tightness, less frictional resistance, and less damage from corrosion. For these reasons lock-bar pipes were adopted for the pipe-line. They were made in lengths of 28 ft., uniformly 30 ins. in diameter and $\frac{1}{4}$ in. thick for heads up to 320 ft., and $\frac{5}{16}$ in. thick for all heads over 390 ft. The pipes were tested under a hydraulic pressure of 400 lbs. per square inch. They were placed in trenches, except at the salt-impregnated beds of the so-called "lakes," where the pipes were laid above ground on trestles and surrounded with galvanized iron, sawdust being placed in the space between the pipe and the galvanized iron in order to prevent the water from freezing.

333. *The Coating.* — The pipe-line is exposed to temperatures ranging from the frosts of winter to 170° F. in summer. As a result of numerous experiments, it was decided to coat the pipes with a composition consisting of one part of asphalt to one part of coal-tar, applied at a temperature of 300° F., and freely sprinkled with sand, while soft and hot, to reduce the risk of the coating running, when exposed in hot weather.

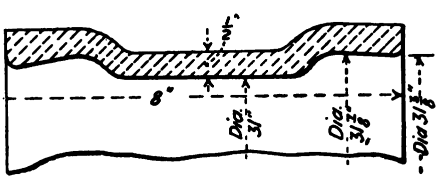


Fig. 205.

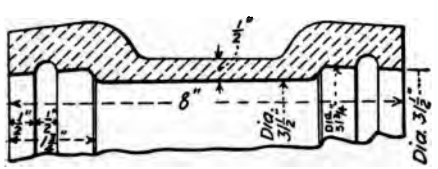


Fig. 205*a*.
Sleeves for Coolgardie Pipe-line.

334. *Joints.* — In order to allow for expansion and contraction, the different sections of pipe were united by simple sleeve-joints, which were poured with lead and caulked by machinery. Each of the joints was a ring, 8 ins. wide, having an inner diameter that was $\frac{1}{2}$ in. larger than the outer diameter of the pipe. For working heads of 320 ft. or less, the sleeve-joint shown in Fig. 205, weighing 126 lbs., was used. For greater heads a stronger joint (Fig. 205*a*), weighing 160 lbs., was adopted. The joints were found to be very water-tight, the leakage in 295 miles being only 480 gals. per mile per day, over 10 months' working.

335. *Appurtenances.* — Gate-valves, 20 inches in diameter, were inserted in the pipe-line, at intervals of about 5 miles, the pipes being connected to these valves by means of reducers. They are actuated by slow-motion gearing, in order to avoid water-hammer. Where long gradients occurred, check-valves were placed in the pipe-line to which they were connected by reducers. Air-valves of the Glenfield pattern were placed at all summits, a nest of three being put at the highest points, a nest of two at intermediate points, and a single valve at the lowest points. These valves are of the double type, provision being made for a large escape of air, when the main is being filled, while air accumulating in the pipe is automatically discharged.

CHAPTER XVI

SERVICE-RESERVOIRS

336. Reservoirs are constructed in connection with water works for the following purposes: (1) to store water; (2) to equalize the flow along an aqueduct or pipe-line; (3) to regulate the distribution of water.

The first kind of reservoirs are large storage basins, which are usually formed by constructing dams across a valley and at depressed points of the ridges on its sides. Large cities have usually a number of such reservoirs, built in the water-sheds from which they obtain their supply. According to circumstances, the quantity of water stored in these basins should be sufficient to supply the city with water during the longest period of drouth that may occur.

337. *Equalizing-reservoirs,* constructed at convenient points along the line of a long aqueduct, insure a uniform flow in the different sections into which they divide the aqueduct, and make it possible to shut off one of these sections for inspection or repair, without emptying the whole aqueduct.

338. *Distributing-reservoirs,* called, also, service-reservoirs, balance the variation in the consumption of water in a community and furnish a supply, in case the water from storage reservoirs, or pumping stations, should be cut off. The capacity of such reservoirs varies from less than 1 to about 10 days' supply. When no ground at the proper elevation for a service-reservoir can be found, and costly stand-pipes may have to take its place, the effective capacity of the storage thus supplied may only be sufficient for a few hours' consumption. If a town is supplied by a duplicate set of pumps, installed within, or near its boundaries, such a small capacity may answer. In such a case, the stand-pipes act mainly as pressure regulators.

When a city is, however, supplied only by one long aqueduct, which is liable to be cut off for inspection or repairs, it is advisab e to provide a storage of 7 to 10 days' supply within the limits of the city, or nearby. Besides storing water and regulating its flow, a reservoir acts as a sedimentation basin, and thus improves the quality of the water.

When the supply consists of surface water, the service-reservoirs are usually open basins, although they are sometimes covered, to protect the water from pollution. For the storage of ground water, or filtered water, the reservoirs should be covered, as solar light and heat cause growths of vegetation in such waters, such as algæ, which give the water a bad taste. Ground water or filtered water may, however, be stored for a short time in open reservoirs without deteriorating in quality, or for longer periods, if it is treated with chemicals.

339. Open Service-reservoirs. — If small, an open reservoir may consist simply of a circular or rectangular basin, having a water-tight bottom and sides, built of concrete, plain or reinforced, according to circumstances. Large open service-reservoirs are usually constructed partly by excavation and partly by embankments, the material for the latter coming mainly from that supplied by the former. Service-reservoirs are generally divided by a central wall into two basins, one of which may be emptied, while the other is kept in service. At each end of the division-wall, a gate-house should be built, one to regulate the flow of the water into the basin and the other to control the out-flow. They should be so constructed that they can be used for either basin. The inlet gate-house may be arranged to act, also, as an outlet gate-house, by connecting it with the distributing pipes. Each basin should have an overflow-weir, constructed in one or both of the gate-houses, or at some other point, and, also, a scour or drain-pipe, controlled by a gate-valve in one of the gate-houses, for emptying it, and the floor of the basin should incline towards the inlet of this pipe so that the whole basin may be properly drained.

340. Covered service-reservoirs* are built to protect stored water from (1) solar heat and light, which cause the growth of vegetation; (2) from atmospheric impurities; and (3) against malicious pollution.

341. *Plan of Reservoir.* — The best plan for the reservoir is evidently a circle, as it has the least perimeter for a given area. A square is the next best plan, as regards economy. Circumstances — such as the necessity of constructing the reservoir within a given piece of land, the desire to avoid rock excavation, etc., — may lead to the adoption of a less economical plan.

The capacity of the reservoir being given, the next problem is to find the most economical depth. As far as the excavation is concerned, the quantity of work to be done is practically the same for any depth that may be adopted. With a view of economy, the reservoir will generally be made partly below and partly above the surface, the earth excavation being estimated to make the required filling around the reservoir, about ten per cent being allowed for shrinkage.

As regards the side walls, floor, roof, and piers, a certain depth can be [illegible]ich gives the minimum cost of construction. The greater the

* Freeman C. Coffin, M. Am. Soc. C. E., Jour. Assoc. Engineering [illegible], XXIII, pp. 1-32.

depth, the less the area of roof and floor, and the length of side walls will be, but, on the other hand, the height and thickness of piers and side walls will increase as the depth of the reservoir becomes greater. The problem of finding the most economical depth for a given capacity of reservoir is too complicated to be solved by simple formulas, and recourse must generally be had to trial calculations.

342. *The side walls* cannot be designed by the formulas used for dams, retaining walls, etc. They are unlike masonry dams, as they are backed with earth, which ought to be made as compact as possible by ramming, watering, etc. They differ from ordinary retaining walls, as their top and bottom are supported, respectively, by the thrust of the roof and the resistance of the floor. If the reservoir is circular, the side walls form an arch subjected to a practically uniform pressure at a given depth. Experience is the best guide in determining the dimension of such walls. The top is generally made $2-2\frac{1}{2}$ ft. thick, which is sufficient to resist the thrust of the roof. In some reservoirs, a steel ring is imbedded in the masonry, near the top of the wall, to increase its resistance to the thrust of the roof. The best manner of making such a ring is of three parts of flat iron, the splices in the different rings being staggered.

The side walls must be made nearly vertical, in order to save excavation, and to keep the reservoir as large as possible. Their thickness must, however, be made greater as their height increases. Circular walls can be safely made thinner than straight walls, on account of their arch resistance. For circular reservoirs constructed entirely below the surface, and not exposed to any great pressure from the outside, thin concrete walls may be used, if steel rings are imbedded in the masonry at different levels, to resist the outward pressure of the water. In order to make the side walls water-tight and smooth, they must be plastered. A good plan is to use two coats of plaster, the first consisting of mortar, 2 parts of sand to 1 of cement, laid on as thick as possible, so as to even up the inequalities of the masonry, and left rough. The second coat should consist of neat cement, $\frac{1}{8}$ to $\frac{1}{4}$ in. thick, and should be nicely smoothed with a trowel.

343. *The floor* should be made of smooth concrete, covering the whole bottom, and plastered. The thickness of the concrete will depend on circumstances. If the foundation consists of compact material, through which but little water is likely to leak, and if there is no upward pressure from the ground water, a thickness of 3 to 4 ins. will suffice. If, on the contrary, the foundation is quite pervious, the floor should be at least 6 ins. thick. This thickness will be sufficient for heads up to 20 ft., if the concrete is properly laid and plastered. If some leakage should occur at first, it will soon stop. The plastering should be done before the cement sets, in order to avoid its pealing off. If some water stands on the concrete, after it has been rammed, dry cement may be spread over it, and smoothed with a trowel. If there is a considerable upward pressure from the ground water, the floor must be formed of inverted arches, sufficiently strong to bear this pressure.

344. *Roof or Vaulting.* — A wooden or metal roof can be used for covering the reservoir, but a masonry vaulting is preferable, on account of its imperishable nature. The latter kind of roof consists usually of brick or concrete arches, which are supported by the side walls and by masonry piers. Concrete is cheaper than brickwork for this purpose, and adapts itself readily to any form of arch. The roof may be formed of ordinary circular arches, but groined elliptical arches offer greater advantages. The latter form of arch requires little masonry, avoids lintel arches or girders for supporting the roof, and provides a clear head-room in each direction.

The strength of the roof arches may be determined by the usual methods, the load assumed being the weight of the arch and the saturated earth filling over it, and, also, the weight of any snow or ice which might rest on the roof. For New England, the weight of snow and ice may be taken at about 50 lbs. per square foot. In some cases, it may, also, be necessary to allow for a load of people standing on the roof. If the rain water is not to be admitted into the reservoir, drain-pipes must be laid to carry it off. One or more iron manholes, with covers about 26 ins. in diameter, must be provided in the roof, for giving access to the reservoir, and one or more ventilator pipes should, also, be placed in the roof.

The centering which must be erected for carrying the roof during construction forms quite an item. For a concrete roof, it is best to place the centering for the whole roof, before any of the concrete is laid. This masonry must be so laid that the thrusts from the sides shall balance each other. With circular brick arches, the same centering may be used several times, but care must be taken not to have any unbalanced thrust in the arch, as it might lead to the collapse of the partially built roof. In all cases, the centering should be left standing until the masonry is sufficiently set — say for about two weeks.

345. *The piers* that support the roof are usually built of brick, stone masonry or concrete. They must be made sufficiently large to bear the pressure they have to support. For brickwork laid in Portland cement mortar, 30 tons per square foot may be taken as a safe load. As the height of the piers increases, they must be reinforced at the bottom, which may be done by offsets. If the foundation of the piers is earth, the bottoms of the piers must be spread so as to reduce the pressure per square foot to what the foundation may safely support. The following table taken from Prof. Baker's "Masonry Construction" gives the loads that various kinds of soil can bear:

TABLE XLVI. — SAFE BEARING POWER OF SOILS

Kind of material	Tons per minimum	Square feet maximum
Clay in thick beds, always dry	4	6
Clay in thick beds, moderately dry	2	4
Clay soft	1	2
Gravel and coarse sand, well cemented	8	10
Sand, compact and well cemented	4	6
Sand, clean and dry	2	4
Quicksand, alluvial soils, etc.	½	1

346. *Appurtenances.* — Every covered reservoir must have an outlet-pipe, and a scour-pipe, controlled by gate-valves, placed near the reservoir. The bottom of the outlet-pipe should be placed 6 ins. or more above the bottom of the reservoir, so as to leave some room for sediment. The scour-pipe should be laid so as to empty the reservoir completely. If the ground water is not to be admitted, a system of drain-pipes must be laid around the reservoir. It may discharge its drainage into the scour-pipe or be connected with a sewer, etc. If the ground water is not objectionable, the drainage system may be connected with a pipe, provided with a check-valve, placed in the floor of the reservoir, and arranged in such a manner, that when the reservoir is full, the check-valve will be closed, while it will open, when the reservoir is empty, so as to admit the ground water, neutralizing thus the inward pressure of the ground water.

Freeman C. Coffin gives the two following tables for making preliminary estimates of the cost of building covered reservoirs.

TABLE XLVII. — COST OF COVERED RESERVOIRS, BUILT WITH ECONOMIC DIMENSIONS

Capacity, U. S. gallons	Round reservoirs			Square reservoirs		
	Diameter in feet	Depth in feet	Cost	Length of side in feet	Depth in feet	Cost
250,000	60.0	12	$4,700	54.5	11	$4,800
500,000	75.0	16	7,800	69.5	14	8,100
750,000	88.0	17	10,500	79.5	16	11,000
1,000,000	98.0	18	12,850	88.5	17	13,550
1,250,000	106.5	19	15,200	99.5	17	16,050
1,500,000	115.5	19	17,550	106.0	18	18,400
1,750,000	120.0	21	19,950	111.5	19	21,700
2,000,000	125.0	22	22,000	118.5	19	22,900
2,500,000	134.0	24	26,200	130.0	20	27,300
3,000,000	144.0	25	30,200	142.5	20	31,450

The above table is based upon the following unit prices.

Item	Cost
Earth excavation	$0.50 per cu. yd.
Rubble in walls, piers, foundations, and floors	6.00 " " "
Concrete in walls, piers, foundations, and floors	6.00 " " "
Brickwork in piers	13.00 " " "
Gravel on roof arches	1.00 " " "
Steel ring	0.05 " pound
Centers, etc., per square foot for total area of reservoir	0.15 " sq. ft.
Plastering walls	0.25 " " "
Plastering floor	0.15 " " "

347. The filters of Albany, N. Y.* (Fig. 206), constructed in 1897 to 1900, are good examples of covered reservoirs, built according to modern plans. They are constructed of masonry — mainly concrete — and covered with earth. The average depth of excavation for the filters was 4 ft., and the material at the bottom was usually blue or yellow clay. The floors consist of inverted groined arches, arranged to distribute the weight of the walls and vaulting over the whole area of the bottom. The outside walls were built of concrete and lined with brickwork, 8 ins. thick. The concrete vaulting was designed for clear spans of 12 ft., with a rise of 2.5 ft. The arches are 6 ins. thick at the crown. The concrete spandrel filling is slightly depressed at the piers. The vaulting was built in squares, the joints being on the crowns of the arches, parallel with the line of piers, each pier being the center of one square. Pipes laid in the piers serve to drain into the filters any rain that falls on the roof. Concrete manholes, built in steel forms, with castings at the top, securely jointed to the concrete, were placed in alternate sections. Above the vaulting, there are 2 ft. of earth and soil, grassed on top.

The following table, condensed from very complete data given by J. H. Gregory, M. Am. Soc. C. E., in *Engineering Record* for Nov. 15, 1913, p. 539, gives data of some groined arches, in the United States and Canada.

* Trans. Am. Soc. C. E., 1900, XXXIII, p. 262.

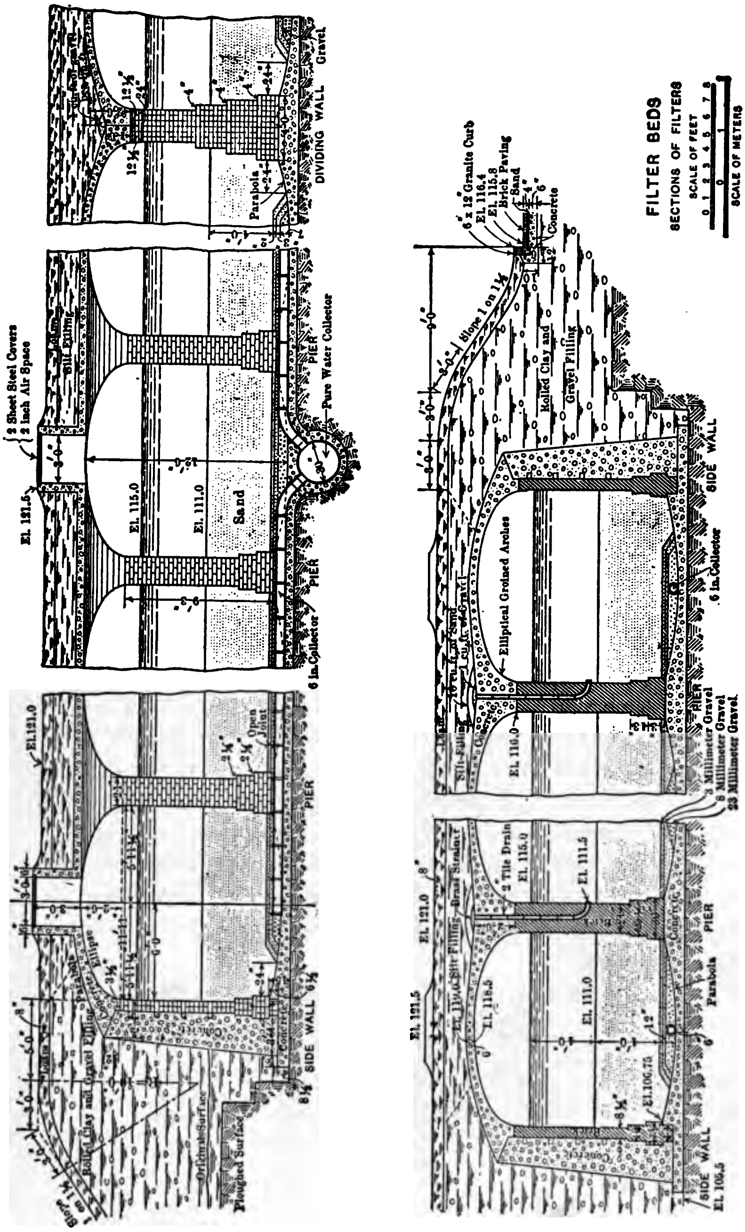


FIG. 206. — Albany Filter-Beds.

TABLE XLVIII.—SOME GROINED CONCRETE ARCHES, COVERING RESERVOIRS AND FILTERS IN THE UNITED STATES AND CANADA

Date	Location	Structure	Depth earth cover, feet	Groined arches span, feet and inches	Rise, feet and inches	Crown thickness, inches	Piers	
							Size, inches	Depression above, inches
1903	Yonkers, N. Y.	Reservoir	4	10–0	1–6	6	$16\frac{3}{4} \times 16\frac{3}{4}$	$9\frac{3}{4}$
1909	Providence, R. I.	Filters	2	10–0	2–6	6	20 × 20	15
1912	Grand Rapids, Mich.	Reservoir	None	10–6 × 10–4	2–9	9	24 × 24	None
1909	New Orleans, La.	Reservoir	1	10–8	2–6	6	16 × 16	......
1909	Springfield, Mass.	Filters	2	11–4	2–0	6	20 × 20	15
1911	Toronto, Ont.	Reservoir	2	11–4	2–0	6	20 × 20	14
1905	New Milford, N. J.	Reservoir	None	11–8 × 9–8	2–6	8	24 × 24	None
1903	Washington, D. C.	Filters	2	11–10	2–6	6	22 × 22	17
1908	Columbus, Ohio	Reservoir	None	11–10 × 9–8	2–6	8	24 × 24	None
1913	Minneapolis, Minn.	Reservoir	None	$11–10 \times 10\frac{1}{2}$	3–0	9	26 × 26	None
1899	Albany, N. Y.	Filters	2	11–11	2–6	6	21 × 21	6
1897	Wellesley, Mass.	Reservoir	2	12–0	2–6	6	24 × 24	None
1900	Superior, Wis.	Reservoir	2	12–0	2–6	6	20 × 20	6
1903	Brookline, Mass.	Reservoir	1.75	12–0	2–6	6	20 × 20	None
1910	Wilmington, Del.	Reservoir	None	12–0	3–0	9	24 × 24	None
1913	Montreal, P. Q.	Reservoir	3	12–0 × 10–3	3–0	8	30 × 30	None
1903	Washington, D. C.	Filters	2	12–2	2–6	6	22 × 22	17
1909	Yonkers, N. Y.	Filters	2	12–2	2–6	6	22 × 22	......
1899	Concord, Mass.	Reservoir	2.5	12–9	3–0	6	24 × 24	None
1907	Torresdale, Philadelphia, Pa.	Filters	2	13–2	3–0	6	22 × 22	21
1908	Pittsburgh, Pa.	Filters	3	13–2	3–0	6	22 × 22	21
1913	Baltimore, Md.	Reservoir	1.75–2.3	13–6	3–0	6	18 × 18	21
1912	Queen Lane, Philadelphia, Pa.	Reservoir	None	13–6	3–9	10	30 × 30	None
1904	Belmont " "	Reservoir	2	14–0	3–0	6	22 × 22	21
1907	Torresdale " "	Filters	2	14–0	3–0	6	22 × 22	21
1909	Springfield, Mass.	Reservoir	2	14–0	2–9	6	24 × 24	18
1905	Washington, D. C.	Reservoir	2	15–6	3–6	6	30 × 30	$24\frac{3}{4}$
1913	Minneapolis, Minn.	Reservoir	2	16–4	3–6	6	20 × 20	24

Since 1850 a number of covered reservoirs have been built in England, Germany, and Austria. Descriptions of a number of these storage basins are given in Proc. Inst. C. E., 1883, LXXIII, pp. 1 to 62, from which the two following are taken.

348. The reservoir at Hampton (Fig. 207) was built in 1880 for the water works of London, England. It is 240 ft. long, 155 ft. wide, 12 ft. deep, and stores about 3,300,000 U. S. gallons. The reservoir is built entirely of concrete. The foundation rests on blue clay, above which the soil consists of excellent clean sharp ballast, admirably suited for

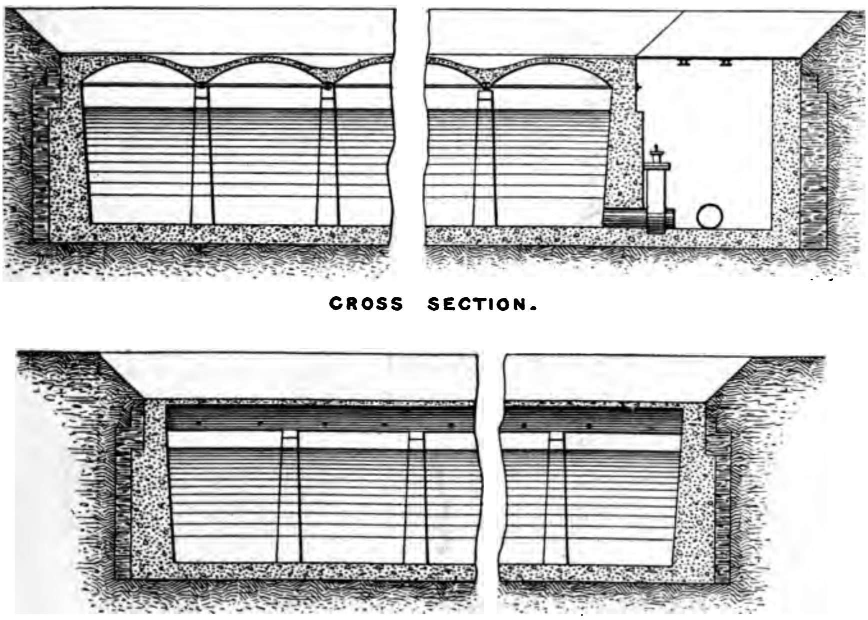


FIG. 207. — Hampton Reservoir, London Water Works.

concrete. The side walls are 6 ft. thick at the base, 3 ft. thick at the top, and 14 ft. high. They are vertical at the back, and are battered on the inner face. The walls are backed with 18 ins. of puddle. The piers are 12 ft., center to center, and 13 ft. high. They are battered, being 2 ft. square at the base, and 18 ins. square at the top. The piers are capped with York stone, on which rest wrought-iron 8-in. I-beams, tied with $\frac{3}{4}$-in. galvanized wrought-iron rods, 6 ft. apart, which fasten the whole roof together, and relieve the side-walls from any outward thrust.

The roof consists of concrete arches, springing from the I-beams, having spans of 12 ft. and a rise of 2.5 ft. The arches are 9 ins. thick at the crown, and 18 ins. thick at the haunches. Six inches of earth is placed on the concrete. The floor is formed of concrete, 1 ft. thick. The reservoir was built entirely below the surface and cost about $40,000.

349. The Burton-on-Trent reservoir (Fig. 208) was built for the water works of the South Staffordshire Water Works Company on a hill south of the town, on stiff clay soil. It was constructed about half in excavation and half in embankment. The reservoir is rectangular in plan, and is covered by brick arches, springing from cast iron girders,

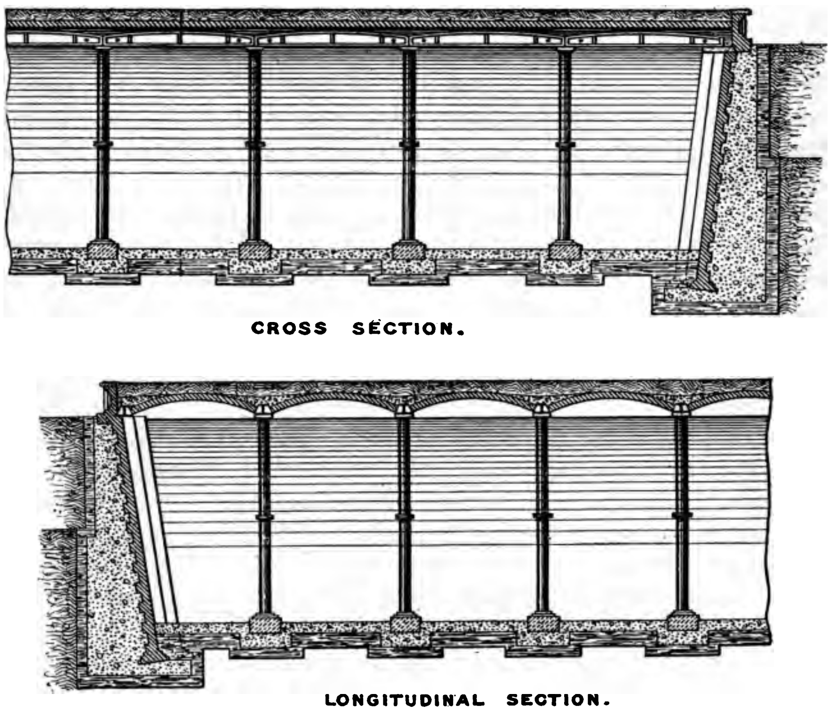


Fig. 208. — Burton Reservoir, London Water Works.

supported on cast iron columns. The dimensions at the floor-line are 224.5 by 145.83 ft., and are, at the top, 229.5 by 150.83 ft. The depth of water is 21 ft.

The external walls are of concrete, faced with brickwork, laid in Portland cement mortar. In plan, the brick walls are formed of a series of brick-piers with curved bays between them. The distance between the centers of the piers is 13.5 ft. in the side walls, and 15 ft. in the end walls. The versed sine of the curves is 1 ft. The brickwork, which is built with alternate courses of four rows of brick, respectively 14 ins. and 9 ins.

thick, has a batter of 1 in 8. It is carried down 3.5 ft. below the floor, and rests with four courses of footings on a foundation of 18 ins. of concrete. The back of the concrete wall is straight in plan, and is carried down vertically, with the exception of a 6-in. offset at the junction with the solid ground and the embankment. The total height of the wall is 25.5 ft. The minimum thickness of the concrete and brick in the center of the bays is 3 ft. $1\frac{1}{2}$ ins. at the top, and 6 ft. $7\frac{1}{2}$ ins. at the bottom. The base of concrete, which extends under the footings of the brickwork, is 11 ft. wide. The concrete wall is backed with, and rests upon, puddle, 12 ins. thick.

The roof is supported by 144 hollow cylindrical columns of cast iron, 10 ins. in external diameter, and 1 in. thick. Each column is made of two pieces, each 10 ft. long, which are bolted together at the center. The foundation of each column consists of a 3- by 3-ft. stone block, 15 ins. thick, which projects 3 ins. above the level of the floor, and is supported on a block of concrete, 5 ft. square by 2 ft. thick, below which there is a 12-in. layer of puddle. Cast iron girders, I-shaped in section, are bolted to the top flanges of the columns. They are 15 ft. long, 10 ins. deep at the ends, and 20 ins. deep at the center. Each girder is bolted at the end to the one next to it, and the girders are tied together by two $1\frac{1}{8}$-in. galvanized tie-bars to each length of girder, and the side walls are thus relieved of any thrust from the roofing.

The covering arches, which are formed of two rings of brickwork, have spans 13.5 ft., with a rise of 18 inches. The spandrels of the arches are filled in with concrete, which is covered with asphalt, to prevent the percolation of rain-water through the roof. This water is collected by drains which discharge at each side of the reservoir. About 1 ft. of soil is laid on the roof and turfed over. The parapet which retains this covering is curved to a height just level with the soil over the arches. It rises 3 ft. above the top of earthwork, and is neatly formed of blue bricks, with panels of blue and red check work. The earth bank is about 10 ft. wide on top, and slopes 4 to 1 to the original surface. The reservoir stores about 4,800,000 U. S. gallons, and cost about $75,000.

350. Covered Reservoirs in Paris, France. — The service-reservoirs of Paris are built with two or three stories, the lower one for river water, for public purposes, while the upper stories contain spring water for domestic consumption. This plan is adopted, owing to the difficulty of finding suitable reservoir sites at the proper elevation within the limits of the city, and, also, to effect a saving in the cost of construction. A description of one of these reservoirs is given below.

*The reservoir of Menilmontant** stores, in two separate basins, spring

* Jour. New England Water Works Assoc., 1888, III, p. 51.

water brought by the Dhuis aqueduct, and river water, pumped from the river Marne. The capacity for spring water was fixed at 2.5 times the discharge of the aqueduct, or at about 26,400,000 gals. The reservoir was founded on the green clay of Montmartre, at a depth of 20–23 ft. from the surface. It has two stories, each of which is divided into two compartments. The two lower compartments, which have a capacity of about 7,500,000 gals., receive water from the river Marne, which is pumped at Saint-Maur. The two upper compartments, which can store about 26,400,000 gals. of water, receive spring water. The reservoir was built with the utmost care in 1863–1866.

The plan of the upper reservoir consists of a semicircle, united with a rectangle.* The diameter of the semicircle, which is 188 meters (617 ft.) long, forms the long side of the rectangle, the other dimension of which is 42.5 meters (139.4 ft.). The outer walls have a thickness of 1.4 meters (4.6 ft.) at the top. The inner batter is 1 in 5 for the portion below the surface of the earth. Above this surface, the exterior face is vertical. The inner faces are also vertical above this level, and are joined with the bottom by an arc of 2 meters (6.6 ft.). The bottom is formed of a series of semicircular groined arches, for the part which covers the Marne water, and of segmental groined arches for the other portions. The intrados of these latter rests upon the marl or clay. The arches are of rubble, 14 ins. thick at the key.

The reservoir is covered by groined arches, composed of two courses of brick, laid flat, which are supported by piers, 2 ft. square, placed 20 ft. between centers. The brick arches are $3\frac{1}{4}$ ins. thick, including the plastering, and are covered with earth and turf, 16 ins. thick. The effluent pipes are one meter (39.4 ins.) in diameter, and can draw water from either compartment of the upper reservoir, or direct from the Dhuis aqueduct. The upper reservoir may be emptied into the Marne storage basin below it, into which its overflow also discharges.

The storage basin for the Marne water occupies only the central portion of the sub-basement of the upper reservoir. It is rectangular in plan, its longer side, parallel with the front, being 104 meters (341 ft.) long, and its smaller side having a length of 90 meters (295 ft.). It is divided into two equal, symmetrical compartments by the division wall, mentioned above. The enclosing walls, which are of rubble masonry, are placed in the middle, and follow the direction of the ranges of arches, supporting the upper bottom. They are 1.2 meters (3.9 ft.) thick, with vertical faces, the interior face being joined with the bottom by an arc of 2 meters (6.6 ft.) in radius. Their top forms a gallery, which allows free passage around the basins. The bottom, which follows the natural

* Debauve's "Manuel de l'Ingénieur des Ponts et Chaussées," 16me Fasc., p. 167.

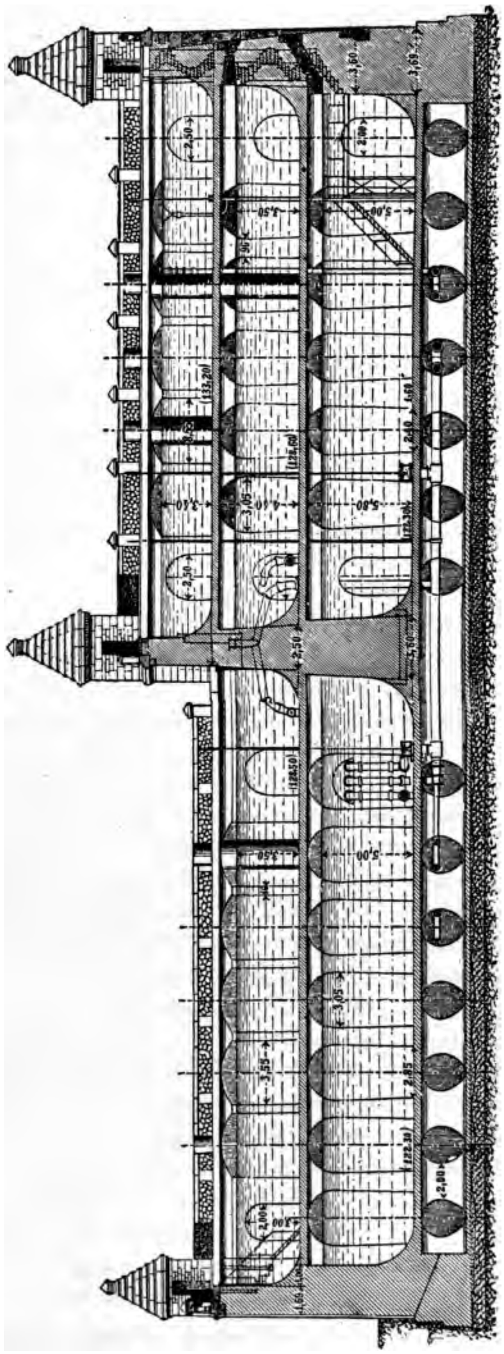


FIG. 209. — Montmartre Distributing Reservoir, Paris Water Works.

surface of the gypsum, is 1 ft. thick. The pillars which support the arches are 1.25 meters (4.1 ft.) square at spring-line, and 1.75 meters (5.7 ft.) at the bottom. The influent pipes can discharge into either compartment. The effluent pipe, which has a diameter of 80 cm. (31.5 ins.), can draw water from either or both compartments.

The Belleville reservoir was built at the highest point in Paris, on the right bank of the river Seine. It has two stories, the upper one, which has a capacity of 6239 cu. m. (1,648,100 U. S. gals.), receiving water from the Dhuis aqueduct, and the lower one, with a capacity of 11,765 cu. m. (3,107,800 U. S. gals.), storing river water for public uses. This reservoir, which was built in a similar manner like that described above, was put into service in 1864.

The reservoir of Montrouge, which has a capacity of 300,000 cu. m. (79,242,000 U. S. gals.), receives spring water from the aqueduct of the Vanne in the upper stories and river water in the lower story. This service-reservoir was built in a similar manner as the storage basin of Menilmontant.

*The reservoir of Montmartre** (Fig. 209) has three stories for spring water and one for river water. The former store 6200 cu. m. (1,637,700 U. S. gals.) of water, and the latter has a capacity of 4800 cu. m. (1,267,-900 U. S. gals.). The water and drainage-pipes, gate-valves, etc., are placed in vaults under the lowest water basin.

* *Ingegneria Sanitaria.* — Igiene delle Abitazioni, Vol. III, "Provvista, Condotta, e Distribuzione delle Acque, dell' Ingegnere," Donato Spataro, Milano, 1893, p. 79.

CHAPTER XVII

STAND-PIPES

351. When a town is situated in a level region, where no point can be found for a reservoir at a sufficient elevation to give the necessary pressure for fire purposes, etc., a stand-pipe, or a tank supported by a trestle, may be used. A stand-pipe is a vertical pipe, made of wrought-iron, steel, or reinforced concrete, resting on a masonry foundation, and built up sufficiently high to obtain the desired pressure.

Stand-Pipes of Wrought Iron and Steel

352. General Dimensions. — If the pipe is merely to form a cushion for a pumping engine, it may have a small diameter; but if it is to provide storage, its diameter may be 50–100 ft. A stand-pipe, built at Wichita, Kansas, was 2½ ft. in diameter and 150 ft. high. The stand-pipe at Youngstown, Ohio, is 100 ft. in diameter and 50 ft. high. These pipes may be regarded as about the extremes as regards diameters.

When a stand-pipe is to be used for storage, its height is usually made about 1½ to 3 times its diameter. As the lower part of a stand-pipe has no value for storage purposes, a tank, made of wood, wrought iron, or steel, resting on a tower or trestle, is often substituted for a stand-pipe.

353. Foundation. — Stand-pipes (Fig. 210) are erected on masonry foundations, which must be carried deep enough, and have sufficient base, to make unequal settling impossible. For stand-pipes up to about 6 ft. in diameter, the base may be formed by a single casting having a groove on top, into which the plates forming the first course are set, and then leaded and caulked, both on the inside and outside. For larger stand-pipes, the base is formed of plates of wrought iron or steel, which should not be less than ¼ in. thick, so as to be able to resist corrosion. The plates are joined together with lap or butt seams, and are connected to the sides of the stand-pipe, either by turning up the edge of the base plate to form a flange, or by means of angles, placed generally on the inside of the pipe, and riveted to the base plate and to the sides. The plates forming the base of the pipe are joined together on blocking, at a convenient height above the foundations. A bed of dry cement mortar, 1 to 2 ins. thick, is spread over the foundation, except under the edges of the stand-pipe, where wet mortar is placed. The base of the pipe is then lowered slowly and evenly onto the bed of mortar, so as to take an even bearing. Any leakage under the base, that may occur when the pipe is filled, will set the dry mortar.

354. Plates. — Stand-pipes are now made almost universally of open hearth steel plates of good quality. The *Standard Specifications for Steel* of the Association of American Steel Manufacturers, which are usually adopted for stand-pipes, are given in the Appendix. The plates are $\frac{1}{4}$ to $1\frac{1}{2}$ in. thick, about 10 to 25 ft. long, and 5 to 9 ft. wide. Most shops can only handle plates up to 1 in. in thickness and 20 ft. long by 8 ft. wide.

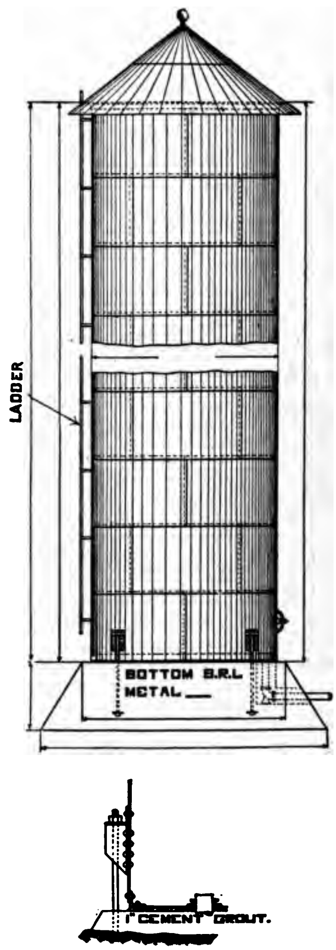


FIG. 210. — Steel Stand-pipe.

The courses of plates are either made cylindrical or with a slight batter. In the former case, the stand-pipe is built up with alternate inside and outside courses, the latter being just large enough to slip over the former. When the courses are battered, the upper edge of each course fits inside of the lower edge of the succeeding course.

355. The joints form always the weakest places in a stand-pipe. Their strength is to be calculated by the formulas given on pp. 104 to 108.

The horizontal joints are generally single-riveted from the top down to a level where the stresses caused by the wind necessitate double-riveting. The vertical joints are designed on an economical basis, single-riveted seams being generally used for plates up to $\frac{1}{4}$ in. in thickness; double-riveted seams for plates $\frac{1}{4}$ to $\frac{3}{8}$ in. thick, and triple-riveted

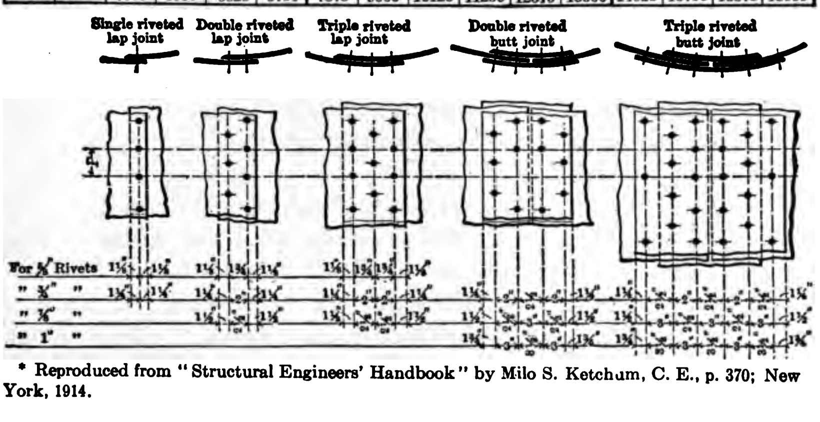
TABLE XLIX. WATER-TIGHT JOINTS.*

	Thickness of plate	Number of rows of rivets	5/8" Rivets: Efficiency of joints in per cent	5/8" Rivets: Pitch of rivets in inches	5/8" Rivets: Effective section of plates	3/4" Rivets: Efficiency of joints in per cent	3/4" Rivets: Pitch of rivets in inches	3/4" Rivets: Effective section of plates	7/8" Rivets: Efficiency of joints in per cent	7/8" Rivets: Pitch of rivets in inches	7/8" Rivets: Effective section of plates	1" Rivets: Efficiency of joints in per cent	1" Rivets: Pitch of rivets in inches	1" Rivets: Effective section of plates
Lap Joints	1/4"	1	48.7	1 7/8	0.121									
		2	**70.7**	**2 1/4**	**0.177**									
	5/16	1	39.5	1 7/8	0.124	47.1	2 1/4	0.147						
		2	65.4	2 1/4	0.205	**70.5**	**3**	**0.220**						
	3/8	2	61.3	2	0.230	66.6	2 5/8	0.250	**70.7**	**3 1/8**	**0.265**			
		3	70.8	2 5/8	0.265	75.6	3 1/8	0.284	73.2	3 1/4	0.274			
	7/16	2				63.5	2 3/8	0.279	66.5	3	0.291			
		3				72.3	3 1/8	0.317	**75.2**	**4**	**0.329**			
	1/2	2				58.9	2 1/4	0.295	63.8	2 1/4	0.319			
		3				69.4	2 7/8	0.347	**72.6**	**3 1/4**	**0.363**			
	9/16	2							61.0	2 5/8	0.344			
		3							70.5	3 3/8	0.397			
Butt Joints	7/16	2				72.0	3 1/4	0.315	72.3	3 3/8	0.316			
		3				82.2	3 5/8	0.359	84.7	3 5/8	0.370			
	1/2	2				72.0	3 1/8	0.360	72.3	3 5/8	0.362			
		3				80.8	3 1/8	0.405	82.8	3 3/8	0.415			
	9/16	2				72.0	3 1/8	0.405	**72.3**	**3 1/2**	**0.407**			
		3				80.5	3 1/8	0.453	82.1	3 5/8	0.463			
	5/8	2				70.7	3	0.442	**72.3**	**3 5/8**	**0.452**			
		3				78.4	3	0.490	81.0	3 3/8	0.506			
	11/16	2				68.3	2 3/4	0.469	**72.3**	**3 3/8**	**0.498**			
		3				75.7	2 3/4	0.522	80.3	3 5/8	0.552			
	3/4	2				66.4	2 5/8	0.498	**70.2**	**3 3/8**	**0.526**			
		3				73.8	2 5/8	0.553	78.0	3 3/4	0.585			
	13/16	2							68.3	3 1/4	0.555			
		3							**75.5**	**3 1/4**	**0.614**			
	7/8	2							66.5	3	0.582			
		3							**74.1**	**3**	**0.647**			
	15/16	2	Note:									70.1	3 3/8	0.657
		3	Heavy figures indicate									**76.5**	**3 5/8**	**0.717**
	1"	2	economical riveted joints									67.3	3 1/2	0.673
		3										**74.7**	**3 1/2**	**0.747**

Note: The distances between rivets at caulked edges shall never exceed 10 times the thickness of plates or straps. The thickness of each strap for butt joints shall never be less than half the thickness of the plates plus $\frac{1}{16}$ inch.

SHEARING AND BEARING VALUE OF RIVETS.

Diameter of rivets, in inches	Area in, square inches	Single Shear at 9000 lb. per sq. in.	Bearing value for different thicknesses of plates, in inches, at 18000 lb. per sq. in.: 1/4"	5/16"	3/8"	7/16"	1/2"	9/16"	5/8"	11/16"	3/4"	13/16"	7/8"	15/16"	1"
5/8	0.3068	2761	2813	3516	4219	4922	5625	6328	7031						
3/4	0.4418	3976	3375	4219	5063	5906	6750	7594	8438	9281	10125				
7/8	0.6013	5412	3938	4922	5906	6891	7875	8859	9844	10828	11813	12797	13781		
1	0.7854	7069	4500	5625	6750	7875	9000	10125	11250	12375	13500	14625	15750	16875	18000



* Reproduced from "Structural Engineers' Handbook" by Milo S. Ketchum, C. E., p. 370; New York, 1914.

lap-seams for plates $\frac{3}{8}$ to $\frac{1}{2}$ in. in thickness. For thicker metal double-riveted, butt-joints are adopted. Standard riveted joints for stand-pipes and tanks are given in Table XLIX.

356. Details. — The top of the stand-pipe should be stiffened against wind pressure by an angle iron riveted on the outside at the top. A manhole should be provided in the lowest ring to permit access to the interior of the pipe for cleaning, etc.

Stand-pipes are usually left open on top. Occasionally they are covered by a roof. Some are entirely enclosed by masonry as a protection against wind and weather. An overflow-pipe is sometimes provided. It should be placed on the outside of the pipe, if ice may form in the pipe. A ladder or a spiral stairway, terminating about ten feet above the ground, should be attached to the outside of the pipe.

357. Anchorage. — Even if the stand-pipe has a large diameter, it is advisable to anchor it to the foundation by means of bolts, which pass through anchor-plates in the masonry, and are attached to brackets, formed usually of angles and plates, riveted to the plates of the first course. In ordinary cases, six to eight anchor bolts are used. The stress on any bolt is calculated by supposing the wind to rotate the pipe on the edge opposite the bolt. As the total pressure of the wind on a cylinder is equal to its pressure on the projection of the cylinder, and as, furthermore, the resultant pressure may be assumed to act at half the height above the foundation, the moment to be resisted by the anchor bolts is:

$$p\,dh \times \frac{h}{2} = \frac{p\,dh^2}{2}, \tag{87}$$

where p = pressure of wind per square foot, assumed usually at 30–50 lbs. per sq. ft.;
d = diameter of the pipe in feet;
h = height of the pipe in feet.

358. Painting the Pipe. — The plates are given one coat of paint at the shop. Before the pipe is filled, the inside should be painted with one or two additional coats of good metallic paint. Some leakage is apt to occur, when the pipe is filled for the first time, but small pin-hole leaks will generally stop in two or three days' time. After the stand-pipe has been found to be in good condition, the outside should be painted with one or two coats of paint. The bottom plate should be painted, before it is placed on the foundation.

359. The inlet-pipe passes through an opening in the masonry, which must be sufficiently large to permit inspection. This pipe, which usually forms also the outlet-pipe, is generally a branch from the force main, which rises by a base-elbow. The inlet-pipe either terminates in a flanged mouthpiece, which is riveted to the bottom plate; or, it passes through a bell-shaped casting, riveted to the bottom plate, in which it is leaded and caulked. The inlet-pipe terminates about a foot above the bottom plate. A special blow-off pipe, flush with the bottom of the pipe, is sometimes provided. Each of these pipes is controlled by a gate-valve, placed near the stand-pipe.

Some kind of a tell-tale must be installed, to indicate in the engine room the height of water in the stand-pipe. An ordinary pressure gauge, attached to the force main, may be used for this purpose, when there is no draft on the force main for service; but if there is such a draft, the tell-tale should consist of a float in the stand-pipe, the position of which may be indicated in the engine room by means of wires, levers, etc. The safest way of indicating the height of the water in the stand-pipe is by means of a mercury column placed in the engine room, when it is not too far away. An electric tell-tale contrivance, which can indicate at a distance of about 20 miles the level of the water in a stand-pipe or reservoir, is described on p. 578.

360. Erection. — The early stand-pipes were erected by means of scaffolding, put on the inside and outside of the pipe. In 1876 an improvement in erecting stand-pipes was introduced. It consists in filling the pipe with water as it is built up, and using a floating stage or derrick for the workmen holding the rivets, which are driven from an outside scaffolding. In later constructions, all scaffolding was dispensed with, the rivets being driven from a float on the inside and held on the outside by workmen suspended in a cage, which was carried by roller-hooks, traversing the top edge of the course of sheets that is being riveted. A still further improvement was made about 1883. In erecting some stand-pipes, a double-decked float was used, the upper deck being occupied by the riveters, while the lower deck was used for the painters.

Owing to the difficulty of obtaining water for filling a stand-pipe as it is erected, the method of erection now usually adopted, is to build a scaffold on the inside of the pipe, and to use a riveter's cage on the outside. The plates are hoisted into place by means of a gin-pole, which is generally bolted to the tank sheets of the ring below the one that is being erected. On large stand-pipes, where scaffolding would be a big item, it is now customary to use a system of brackets on the inside of the stand-pipe, which are bolted into the open holes in the ring below, and moved up, as the work progresses. In this case, a riveter's cage is also used on the outside.

361. Failures. — There have been many partial or complete failures of wrought iron and steel stand-pipes. Prof. Pence has described these failures and discussed their causes.* Out of sixteen total failures from 1859 to 1894, two were due to defective masonry or foundation, one was caused by overturning, and the remaining cases originated in, or were accompanied by, plate fracture. Nine of thirteen stand-pipes in which plate fracture occurred were made of steel, and the remaining four were built of wrought iron. When steel was first used for stand-pipes, tank steel — an inferior quality of the metal — was largely used. This accounts for some of the failures of steel stand-pipes. Of late years, however, these structures have been made of a good quality of open hearth steel.

362. Specifications. — C. W. Birch-Nord has prepared a very complete set of specifications for steel stand-pipes and elevated tanks,† which have been largely used. Most manufacturers of steel stand-pipes and tanks have up-to-date specifications for such works, prepared by their engineers.

Stand-Pipes of Reinforced Concrete

363. Reinforced concrete is an excellent material for stand-pipes, providing it is made water-tight. It reduces the cost of maintenance to a minimum, and gives a stand-pipe a finer appearance than steel does. In most stand-pipes of reinforced concrete, more or less trouble has been experienced from leaks, but this is, doubtless, due to the comparative novelty of this kind of construction, and, without doubt, efficient ways will be devised for making such structures water-tight.

A stand-pipe, 10 ft. in diameter and 43 ft. high, built in 1899 in the filter house of the East Jersey Company at Little Falls, N. J.,‡ is probably the first reinforced concrete stand-pipe built in America. Its wall is 15 ins. thick at the bottom and 10 ins. thick at the top. Since then, the number of structures of this kind in various parts of the United States has increased rapidly, as will be seen from Table L.

364. Design. — In designing stand-pipes of reinforced concrete, reliance is placed entirely on the steel reinforcement to resist the horizontal pressure of the water, and the foundation is made strong enough to carry the weight of the structure and of the water. In most stand-pipes cracks occur in the masonry, near the bottom, especially on the south side of the tank, which is subjected to greater changes of temperature than the north side. These cracks cause leakage, and, in cold

* "Stand-pipe Accidents and Failures," by Wm. D. Pence, C. E., New York, 1895, p. 139.

† Trans. Am. Soc. C. E., 1909, LXIV, pp. 548–563.

‡ Trans. Am. Soc. C. E., 1903, L, p. 454.

weather, lead to the scaling off of some of the concrete at the outer surface of the stand-pipe. Even when the leaks appeared to be stopped by various methods, the emptying of the stand-pipe for a short time, and subsequent refilling, often starts the leakage again.

Various methods of remedying this trouble have been suggested. In a paper on "A New Theory for the Design of Reinforced Concrete Reservoirs," * Hiram B. Andrews,† M. Am. Soc. C. E., advocates:

1. The use of an extra-rich composition of concrete, mixed 1 : $1\frac{1}{2}$: 3, or 1 : 1 : 2, in order to make it water-tight.
2. Increasing the thickness of the concrete wall to such an extent that the tensional stress in the concrete will not exceed a safe limit.
3. Putting in a vertical steel reinforcement, especially in the lower part of the wall.
4. Making the joint between the base and the wall strong, by turning the steel reinforcement of the former well up into the latter.
5. Placing a thin steel ring in each horizontal joint, between days' work, in order to prevent leakage at these joints.

William Mueser, M. Am. Soc. C. E., recommends exactly an opposite course,‡ *viz.*, to separate the base entirely from the wall of the pipe, so as to leave the latter free to expand and contract under changes of temperature and varying water pressure, and to fill the joint between the bottom of the tank and its wall with an elastic substance, to insure water-tightness. He has carried out this plan successfully in a stand-pipe, 60 ft. in diameter and 48 ft. high, at Sanderson, Texas, and in another at Fulton, N. Y. (p. 371).

Various expedients have been tried to make leaky concrete stand-pipes water-tight. About every known method of waterproofing was tried in the stand-pipe at Attleboro (p. 364). The method finally resorted to was the placing of an elastic membrane, composed of layers of felt and a cementing compound or hot asphalt. Some engineers think that water-tightness may be secured by using a richer mixture of concrete — say 1 : 1 : 2.

To illustrate the construction of stand-pipes of reinforced concrete, we give below detailed description of some important works of this kind.

365. The concrete stand-pipe at Attleboro, Mass.,§ was built in 1904 by the Aberthaw Construction Company, of Boston, Mass., to provide additional storage for the public water works. This tank is 50

* Jour. Assoc. Engineering Societies, 1911, XLVI, p. 391.

† Engineer for Simpson Bros. Corporation, Boston, Mass.

‡ Trans. Am. Soc. C. E., 1911, LXXIV, p. 392.

§ Jour. New England Water Works Assoc., 1906, XX, p. 302; Jour. Assoc. of Engineering Societies, 1911, XLVI, p. 402.

ft. in diameter and 100 ft. high to the level of the overflow. It was designed on the basis that the steel reinforcement should be able to resist the whole horizontal pressure of the water, without being stressed more than 13,500 lbs. per sq. in. This reinforcement consists of steel bars, $1\frac{1}{2}$ ins. diameter, which are placed in two rows for a height of 61 ft. from the floor. For the next 20 ft., only one row of $1\frac{1}{2}$-in. bars is used, and, for the remaining distance to the top of the stand-pipe, the wall is reinforced by one row of bars, $1\frac{1}{8}$ ins. in diameter. The top 15 ft. of the wall has the same reinforcement, in order to provide sufficient strength to resist ice pressure, which was assumed in the calculations at 30,000 lbs. per sq. ft. The computations showed that with combined water and ice pressure, the steel would not be stressed over 18,000 lbs. per sq. in. As a matter of fact, no trouble from ice pressure has been experienced in this tank, owing to the frequent fluctuations in the level of the water surface.

366. *Construction.* — The wall of the stand-pipe, which is made of 1 : 2 : 4 concrete, begins at the bottom with a thickness of 18 ins., and is tapered to 8 ins. at the top. During the construction, the steel bars were supported by 4-in. channels, placed vertically about 15 ft. apart. The steel bars rested on $\frac{1}{4}$-in. rods, put in $\frac{3}{8}$-in. holes, which had been punched through the flanges of the channels for the desired spacing, and the ends of the rods were bent up so as to hold the bars firmly. In the upper part of the wall, where only one row of steel was placed, 3-in. channels were used as spacers. Each hoop consists of three bars, having a lap of forty diameters at each joint, and, as an additional precaution, the bars are firmly held together at each joint by two Crosby wire rope clips. Tests made in the Watertown Arsenal have proved that two of these clips will hold the steel bars tightly enough to secure the full working stress of the bars. The joints of the hoops are staggered. No vertical steel is used.

The top surface of the floor is reinforced with $\frac{1}{4}$-in. square twisted bars, placed 6 ins. between centers, both ways. These bars are carried well up the curved corner and into the wall of the stand-pipe. At intervals of about 3 ft. around the circumference of the stand-pipe, $\frac{5}{8}$-in. square twisted bars are placed radially, their ends projecting up into the wall for a height of 10 ft.

The foundation consists of a slab of concrete, 18 ins. thick, except immediately under the wall, where the foundation is 4 ft. deep for a width of 5 ft. A concrete curb, 12 ins. thick and 3 ft. high, with a curved top, is built around the bottom of the stand-pipe, but is not monolithic with the foundation slab.

The stand-pipe is surmounted by a Guastavino tile dome, provided with suitable means for ventilation. A gate-house, enclosing the gate-valves, and giving access to the interior of the stand-pipe through a passage, covered by a balanced manhole cover, is built on one side. The overflow is formed by a series of rectangular slots provided in the wall. The aggregate area of these openings is equal to that of the inlet-pipe.

367. *Laying the Concrete.* — The concrete was laid in the Attleboro stand-pipe from a large tower, built, on the inside of the tank, of 8 × 8-in. hard pine timbers, properly braced. Two bull-wheel derricks were placed on top of this tower. The concrete was handled in a bottom dump-bucket, which was swung into the desired position by the derricks, dumped into boxes, and shoveled into the wall. Wooden forms, about 7 ft. high, were used. They were made of such size that they could be easily handled by the derricks, or, in case of necessity, by the men themselves.

368. *Making the Concrete Water-tight.* — When the stand-pipe was put into service in December, 1904, it was not found to be absolutely water-tight, as required by the contract for its construction, and although the leakage was at first only trifling, it caused, on account of the cold weather, a scaling off on the outer surface of the stand-pipe at certain points, beginning five feet from the bottom and extending about ten feet upwards. Various expedients for stopping the leaks * were tried by the contractors, who were obliged to maintain the tank water-tight for one year. In 1906 the entire inner surface of the stand-pipe was thoroughly cleaned, and was then picked, to insure a good bond with the cement plaster, with which the inside of the wall was then coated. The first coating consisted of a mixture of one part cement, one part sand, and 2 per cent of lime. Three more coats of 1 : 1 cement mortar, without the addition of lime, were applied to the inner surface. Each coat was floated, until a hard, dense surface was produced, which was then scratched to make a bond with the next coat. Although this work was done by expert workmen, it did not stop the leakage entirely.

While the inner surface of the stand-pipe was being plastered, the outer surface was repaired by digging around the outside row of steel reinforcement, putting on iron clips, made of $\frac{3}{4} \times \frac{1}{8}$-in. iron, bolted through, and then cement was forced into the cavities around the clips, by throwing it a distance of 4 or 5 ft., to insure filling the voids. This process was continued until the cement covered the entire outer surface, so that further plastering could be perfectly bonded. Expanded metal was placed on this surface, and was forced over the clips that stood out horizontally. The surface of the metal was covered with a coat of plaster, which was carefully troweled, and then a coat of metal was

* Jour. New England Water Works Assoc., 1915, XXIX, p. 179.

placed outside of that plastering, the ends of the clips being turned at right angles to hold the same in place. The work was completed by a final outside coat, making thus a very firm and compact surface, equal to that of any part of the structure.

369. *Sylvester Process.* — As the work described above did not stop the leaking, the "Sylvester process" was next tried. This process was used successfully in 1870 in making the brickwork of one of the gatehouses of the Croton Water Works water-tight,* and has since been used with good success on reservoir walls that were not exposed to a greater head than about 40 ft. In order to give the process a trial, four coats of the mixture were applied to the bottom 35 ft. of the inner surface of the stand-pipe. The mixture was prepared according to the following formula.

"Dissolve ¾ lb. castile soap in one gallon of water. Dissolve 1 lb. pure alum in eight gallons of water. Both must be thoroughly dissolved. Before applying to the walls, the surface must be perfectly clean and dry; temperature must be about 50° F. First, apply soap at boiling temperature with a flat brush, taking care not to form a froth. Wait 24 hours so that the solution will become dry and hard upon the walls, then apply the alum in the same way, at a temperature of 60 to 70° F. Wait 24 hours, and repeat with alternate coats of soap and alum."

After four coats of the dissolved castile soap and alum had been applied, the stand-pipe was filled with water for the full height of 100 ft. Under this pressure, only four leaks appeared in the part which had been treated by the Sylvester process. The whole inner surface, from top to bottom, was then given four coats by the Sylvester process. The result was very satisfactory, although the stand-pipe was not absolutely tight. The contractors applied five more coats of the Sylvester mixture to the whole inner surface, making thirteen coats in all for the lower 35 ft. of this surface. This treatment seemed to make the stand-pipe practically tight and it was accepted by the water commissioners on December 6, 1906. Some wet spots that appeared on the outer surface were attributed to condensation of the atmosphere. Later a few leaks occurred.

370. *Elastic Membrane.* — The Sylvester process did not stop leakage from the stand-pipe permanently. This may have been due to the fact that the various coatings, applied on the inside of the stand-pipe, became hard and brittle and were cracked by contraction and expansion in the concrete. In this stand-pipe, as in most others, leakage is especially apt to occur in the south side of the stand-pipe, which is exposed to greater variations of temperature than the north side. After

* Trans. Am. Soc. C. E., 1870, I, p. 203.

consulting with various experts on waterproofing, it was decided to put an elastic membrane on the inner surface of the stand-pipe,* similar to the one used for the Westerly stand-pipe. The elastic membranc was composed of felt and cementing compound or hot asphalt, permanently elastic, built up by successive layers, the number varying according to the water pressure. In this manner a permanent elastic waterproof shield was formed. The work, which was similar to the waterproofing applied to bridges, etc., was done as follows:

The tank was thoroughly cleaned and allowed to become perfectly dry. The bottom and the walls, as high as a man could reach, were then brushed with wire brushes, to remove all sediment and to obtain as clean a surface as possible. Next these surfaces were painted with a special preparation, and then the first coat of hot asphalt was applied. After the required coats of asphalt and felt had been put on the bottom and on the walls for a height of about 6 ft., a floating stage was built in the tank, and about 5 ft. of water was let into the tank to float the stage. The melting tanks were placed on top of the stand-pipe, so that the hot asphalt could be delivered quickly on the stage. Five layers of felt, and six of compound, were used to make the elastic membrane. About 5 ft. of wall per day was treated as described above, and tested by admitting water into the stand-pipe. A band of iron was placed around the inner surface at the top, to prevent the elastic membrane from becoming loose.

371. *Outside Lining of Brickwork.* — The outer surface of the stand-pipe, which was being injured by the scaling off of concrete, was protected by building an 8-in. brick wall around the tank, from the foundation to the top. The air space between the brickwork and the concrete keeps the latter from undergoing great changes of temperature. Weepers were left in the brickwork, near the bottom, to provide an outlet for any leakage that might occur.

The treatment on the inside and outside of the tank is said to have stopped all leakage.

372. The concrete stand-pipe at Westerly, R. I.,† Fig. 211, was built in 1910, to provide additional storage for the water supply of Westerly and some neighboring communities. It has an inner diameter of 40 ft. and a height of 70 ft. from the floor to the overflow. The height from the ground to the top of the ventilator is about 88 ft. The walls

* The contract for this work was let for $3000 to Bird & Sons, Walpole, Mass., who had done similar work in the Westerly stand-pipe.

† Jour. Assoc. Engineering Societies, 1911, XLVI, p. 405; Trans. Am. Soc. C. E., 1911, LXXIV, p. 375; Jour. of the New England Water Works Assoc., 1915, XXIX, p. 169.

are 4 ft. thick at the floor line, tapering to 14 ins. at a point 5 ft. from the floor, and continue with this thickness up to the water-line. Access to the interior of the stand-pipe is provided by a chamber 13 ft. long by 4 ft. wide, having a square manhole opening 2 by 4 ft., on the outside, and a 30-in. round iron manhole cover on the inside floor. The roof, which has a diameter of 41 ft. and a rise of 13 ft., is a Guastavino dome of dark red glazed tile. A steel ladder, 1 ft. wide, is attached, on the outside of the stand-pipe, to bronze bolts in cast-iron sockets, with 1-in. bronze faces, which are set into the wall at intervals of 16 ft.

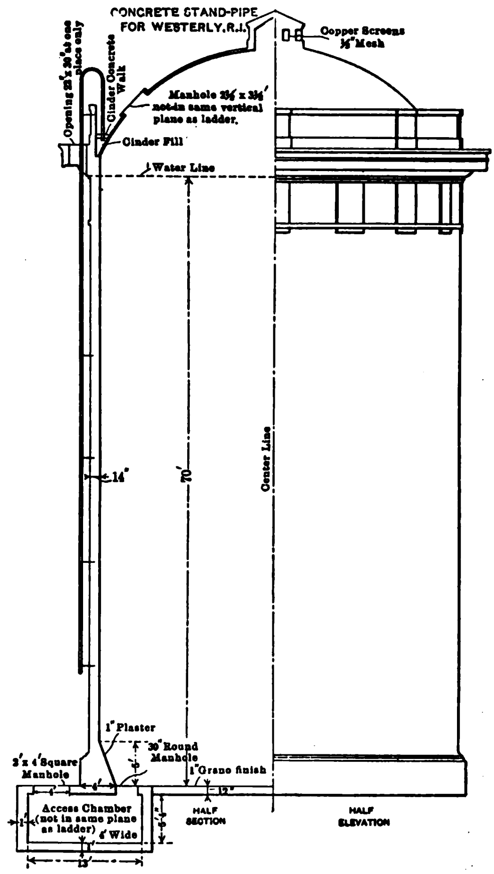


FIG. 211. — Stand-pipe of Reinforced Concrete, Westerly, R. I.

373. *The steel reinforcement*, placed in the concrete, is as follows: ¼-in. square rods are put in the floor, 6 ins. between centers, both ways. These rods are placed 1 in. below the floor surface and are bent up 4 ft. into the wall. The main reinforcement consists of plain, round, mild steel bars, 1½ and 1⅛ ins. in diameter. These bars are 71 and 69 ft. long, so that two are required to form a complete ring, allowing forty diameters for lap, and they are held in place by two Crosby clips at each lap. During the construction, the bars or hoops were supported by twelve 1½-in. vertical iron pipes, placed 30 degrees apart, and resting on standard 6-in. flanges on the floor. These pipes were coupled in 3-ft. lengths, as each successive form was set up. To secure the required spacing of the hoops, ¼-in. holes were drilled in the vertical pipes, at the proper intervals, and hooks were placed in these holes, upon which the hoops rested, until embedded in the concrete.

In order to prevent cracks in the masonry, near the base, like those that occurred in the Attleboro stand-pipe, more steel reinforcement was placed near the base of the wall than in the higher parts. For this purpose the tensile stress allowed in the lowest foot of the wall was limited to 6000 lbs. per sq. in., and this limit was increased 1000 lbs. per sq. in. for each foot rise of the wall, until the stress of 12,500 lbs. per sq. in. was reached. This limit was used for the rest of the wall. No vertical steel reinforcement was used.

Special reinforcement was put at some other parts of the stand-pipe as follows: $\frac{5}{8}$-in. round and $1\frac{1}{4}$-in. square rods around the manhole and roof of the access chamber; $\frac{1}{4}$-in. rods, set horizontally, and $\frac{5}{8}$-in. round rods, 4 ft. center to center, under the dome seat, and bent out into the cornice; and round $\frac{5}{8}$-in. rods, 7 ft. long, placed 2 ft. between centers, and run vertically up into the parapet wall.

374. *The concrete* consists of vulcanite cement, good bank sand, and crushed granite, mixed approximately in the proportion 1 : $1\frac{1}{2}$: 3. These proportions were found by careful experiments to give the concrete its maximum density. Five per cent by weight of limoid, a form of hydrated lime, was added to the cement as a waterproofing, but the main reliance in this respect was placed upon the density of the concrete. Special care was taken to secure good joints between the different days' work. The top surface of the concrete was thoroughly cleaned each night, after it had taken its initial set, and all the laitance was removed, leaving clean sand and stone surfaces exposed. In resuming work the following day, the concrete surface was covered with grout, and a 1-in. layer of 1 : 3 mortar was put in, before the concrete was poured. Owing to the brand of cement used, and to the addition of the hydrated lime, the concrete has a white color.

The floor has a 1-in. granolithic finish, which was carried up as a plaster coat to the top of the inside level. The outside surface was rubbed with carborundum, and then painted with a grout wash, except at the base, which was picked, from the ground up to a level, 2 ins. below the top of the molding. The overflow consists of 3 × 12-in. holes, spaced 45 degrees apart, which were left in the masonry at the high-water level.

375. *Leakage.* — When the tank was filled for the first time, several porous spots showed dampness on the outside. They were quite wet in the morning, but most of them were dried up by the sun. They were made water-tight by forcing grout into the concrete at these spots, under a pressure of 100 lbs. per sq. in. Another leak appeared along the top of the molding on the outside. It was caused by a horizontal crack, which extended nearly half way around the circumference, varying in amount of leakage. An attempt was made to stop this leak by heating the wall for 8 ft. above the floor with charcoal furnaces, and then applying hot paraffin. Although several coats of this material were put on, the leakage was not entirely stopped.

376. *Method of Construction.* — The manner in which this stand-pipe was constructed differed much from the methods employed in the Attleboro tank. The tower required for hoisting the concrete was built outside of the stand-pipe, so as to clear the cornice by about a foot. It was constructed of 6 × 6-in. uprights, thoroughly cross-braced, and was made large enough for a 1-yd. Ransome auto-dump bucket. A No.

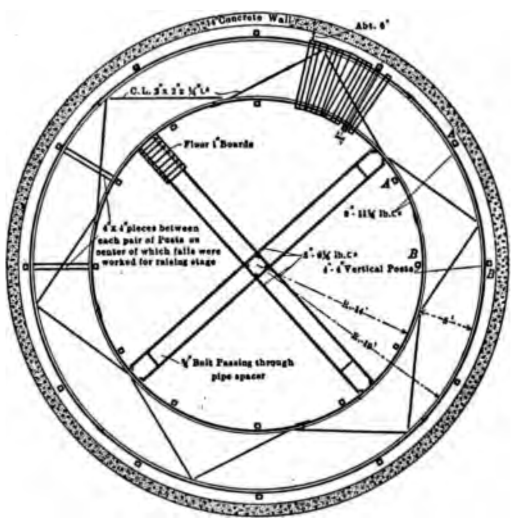


FIG. 212. — Diagram of Inside Stage for Concrete Stand-pipe.

2 Smith concrete mixer was placed in a pit at the tower, and the materials required for the concrete were dumped from the ground into the hopper of the mixer. The concrete for the foundation, floor, and base was hoisted about 20 ft., and dropped into a chute which carried it to the center of the tank. From this point it was delivered through a movable section to the desired place.

A movable stage (Fig. 212) was placed inside of the stand-pipe and was raised gradually by means of differential pulleys, as the work progressed. It was made of two rings of 8-in. channels, which were braced by 2 × 2 × $\frac{1}{2}$-in. angles, and was covered with 2-in. planks, 6 ft. long. The inner ring was braced on two diagonal lines, at right angles to each other, by two pairs of 5-in. channels. One pair was covered with 1-in. boards, 2 ft. long, to form a space for storing wheelbarrows and other tools. The stage, which cleared the masonry wall by about 6 ins., was supported by twelve pairs of 4 × 4-in. uprights, which were put up in 16-ft. lengths as the wall was raised. Each pair of uprights was well braced and had two 1 × 6-in. ledges on which the stage rested (Fig. 213). The outside posts went down vertically to the point where the wall began to flare in and there they bent toward the center and rested on the bottom, close to the wall. The bottoms were braced apart by horizontal, radial struts, and the inside posts were all diagonally braced in radial planes down to these struts.

Each pair of 4 × 4-in. posts had a 4 × 6-in. cap or cross-head, provided with a wire hook at its center. A corresponding hook was fastened to a

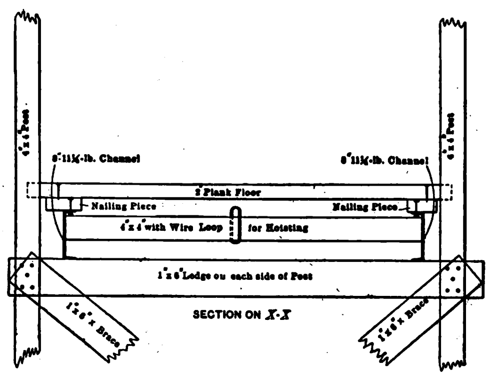


Fig. 213. — Method of Supporting Stage Floor.

4 × 4-in. piece of timber, placed parallel with the ledges, and wedged in radially between the 8-in. channels. The stage was raised by means of eight 2000-lb. differential blocks and chains, attached to the above-mentioned hooks, each being worked by two men. The whole floor was moved, as a unit, in a few moments. As soon as the proper height was reached, new sets of ledges were nailed to the uprights, to support the stage. In order to make the work less hazardous, the water was allowed to rise in the tank, as the work progressed, and was kept about 20 ft. below the stage.

The concrete for the wall was hoisted in the bucket, and dumped into a hopper, hung on the face of the timber tower, about 3 ft. above the stage. A platform, 6 ft. wide, was laid down over the top of the wall, between the stage and the tower. A gate in the bottom of the hopper served for supplying the wheelbarrows, which were wheeled around and dumped directly into the forms, the stage being kept flush with the top of the form that was being poured.

The forms for the outside and inside of the base of the stand-pipe were made of wood, but those for the plain wall were all made of $\frac{1}{8}$-in. steel plate, in panels about 3 ft. deep and 8 ft. long, the edges being strengthened by $1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{8}$-in. angles, which were riveted on the plates. Each panel had, also, two vertical stiffener angles. The angles on the edges were bored for bolts, which served to hold the panels together. Two men could easily handle one of the panels, and two complete sets were kept. The steel forms gave the concrete a very smooth finish. The inside forms were similar to those used on the outside of the wall. The tank has a capacity of 660,000 gals. It cost $17,961 or about $2\frac{3}{4}$ cents per gallon.

377. *Engineers.* — The tank was designed by the Aberthaw Construction Company, and Samuel M. Gray, M. Am. Soc. C. E., acted as Consulting Engineer for the water company.

378. Concrete Stand-pipe with Sliding Base at Fulton, N. Y.* — A reinforced concrete stand-pipe, having an inner diameter of 40 ft., and a height of 100 ft., from the bottom to the overflow, was built in 1913, according to the plans of William Mueser, M. Am. Soc. C. E. It differs from the usual plan adopted for concrete stand-pipes, by having a slip-joint introduced between the base and the wall of the stand-pipe, an improvement in the design of such works suggested by Mr. Mueser, in 1911,† for a concrete reservoir at Sanderson, Texas, and sometime previously for a four and one-half million gallon reservoir at Baltimore, Md. In most of the reinforced concrete stand-pipes, constructed prior to 1911, shear-cracks had occurred in the masonry, near the base, due to changing water pressure. Mr. Mueser claimed that this was due to the fact that the rigid concrete base prevented the walls near the bottom from expanding and contracting, and he thought that this trouble could be avoided by a flexible joint between the base and the wall.

379. *Design.* — In the stand-pipe at Fulton, N. Y., the bottom is designed to carry the load of the water, and the wall to resist the horizontal water pressure. These two parts of the stand-pipe are separated from each other by an elastic joint (Fig. 214). The concrete base is troweled at the joint to a smooth finish, then plastered with a $\frac{1}{8}$-in. coating of graphite paste, and covered by a 20-gauge copper sheet, on which the wall rests. Leakage at the joint is prevented by a trapezoidal asphalt wedge, 8 ins. high, on the inside of the wall. This joint will always remain tight, if the mastic has sufficient flexibility. The greater the pressure, the tighter the joint will be. If the asphalt seal should prove to be of bad quality or become brittle, in course of time,

* *Eng. Record,* Jan. 10, 1914, p. 43.

† Trans. Am. Soc. C. E., 1911, LXXIV, p. 392.

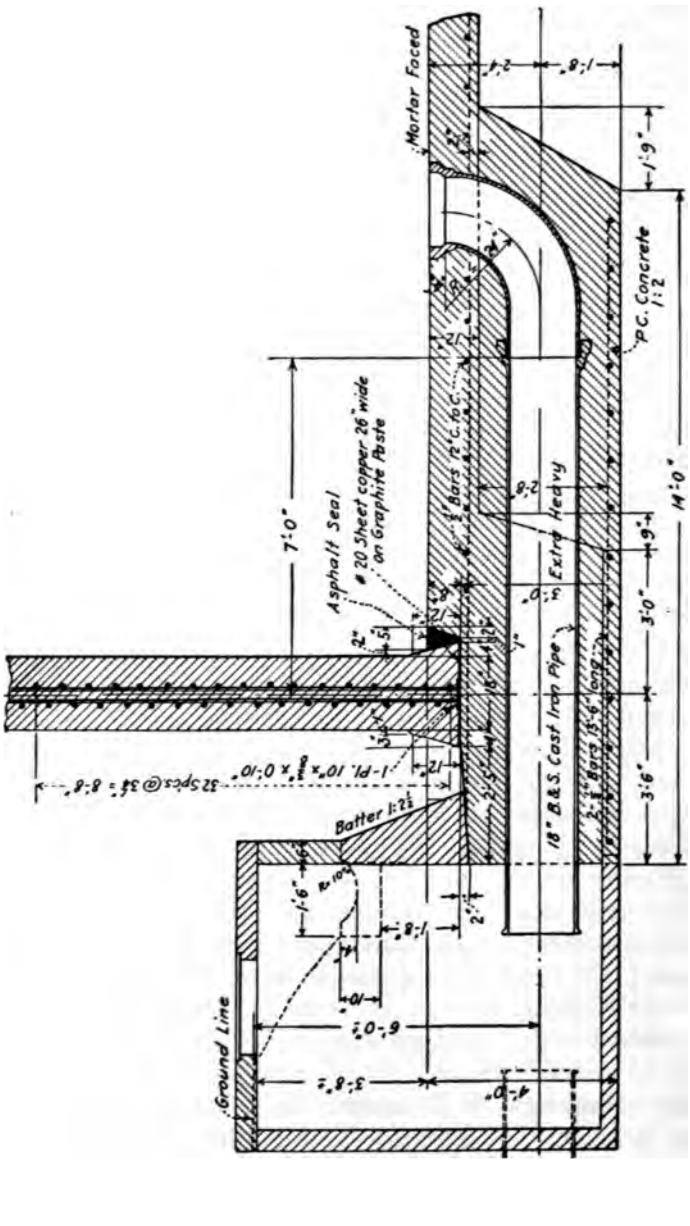


FIG. 214.—Sliding Base of Concrete Stand-pipe at Fulton, N. Y.

it would be an easy matter to scrape out the V-joint and to fill it with good material.

In order to facilitate the form work, the wall was made 18 ins. thick throughout. It is reinforced by *Diamond bars*, $1\frac{1}{2}$ and $1\frac{1}{4}$ ins. in diameter, spaced from $3\frac{1}{4}$ to 9 ins. between centers, the rings being placed in two vertical planes, to facilitate the erection and distribution of the steel in the wall. Z-shaped struts, each made of two $3 \times 3 \times \frac{1}{4}$-in. angles, and spaced equal distances apart on a circle of $41\frac{1}{2}$ ft., in the center of the wall, serve to support the reinforcement, which is held in place by $\frac{1}{4}$-in. round clips. The joints in the reinforcement are made by overlapping and splicing with two $\frac{3}{4}$-in. round clamps. These joints are staggered at least 10 ft. for two consecutive rings. Three bars are used for each ring, and the overlap is made 51 ins. for the $1\frac{1}{2}$-in. bars, and 39 ins. for the $1\frac{1}{4}$-in. bars.

In addition to the supporting struts, there is a vertical reinforcement, consisting of seventy $\frac{3}{4}$-in. bars, seven between each pair of struts, spaced equal distances apart. Forty of these bars reach only to a height of 30 ft. The overflow is formed by six openings, each 18 ins. wide, placed in the wall, 100 ft. above the bottom of the stand-pipe.

The stand-pipe is surmounted by a reinforced-concrete dome with a rise of 10.75 ft., which is topped with a lantern. The total height of the structure is 117 ft. 1 in., from the lowest to the highest point. Access to the interior of the stand-pipe is obtained by a steel ladder, which is placed on the outside of the stand-pipe, is bent around the coping, and descends downward on the inside through a manhole in the roof. The ladder is supported on the top of the wall, and bolts, screwed into bronze sockets, embedded in the concrete, serve as spacers.

380. *Construction.* — The foundation of the stand-pipe consists of gravel with very fine sand in alternating, inclined and not uniform layers. After the foundation excavation was made, the reinforcing bars were placed, and the concrete was deposited. No forms were used, except a rim, forming the slope of the groove for the asphalt. This form was made of two horizontal boards, spaced 8 ins. apart, and provided with a facing of plaster of Paris. The foundation was slushed with grout, and the concrete, mixed 1 : 2 : 4, was then laid without interruption and finished with a wearing surface, 1-in. thick, composed of 1 part cement and $1\frac{1}{2}$ parts of crushed granite, and troweled to a smooth finish. After the concrete had set for four days, a paste of graphite and mineral oil was spread over the joint between the bottom and the wall, and pressed hard into the concrete, making a finishing layer, $\frac{1}{8}$-in. thick. Upon this anti-friction paste, the copper ring of No. 20 gauge, approximately $\frac{1}{32}$ in. thick and 26 ins. wide, was then laid.

In order to avoid leakage at horizontal joints in the wall, the concrete was laid continuously, day and night, by three shifts of men. This was made possible by the use of sliding forms, which were raised on the inside and outside of the wall, as the work progressed. The forms were 5 ft. 4 ins. high, and the outside and inside parts were rigidly connected overhead by twenty frames, spaced equally apart. The inner forms were connected with the working platform, which they supported. The platform and inner forms were, therefore, raised together. A jacking arrangement, consisting chiefly of a movable screw and a stationary nut, was provided at each of the twenty frames or cross-connections. This screw had a $3\frac{1}{2}$-ft. extension, inserted in a gas pipe, and supported at its top on a washer.* When the form had been moved up far enough to require a longer pipe, the screw was turned up, and a new 3-ft. section of gas pipe was inserted. A capstan-nut, supported by a steel plate, was placed above the nut on the screw. By turning the capstan-nut a quarter of a revolution, by means of a short bar, the form at that point was raised about $\frac{1}{8}$ in. One man, going around and turning each of the capstan nuts a quarter or half a turn, kept the forms in a steady motion upward.

No waterproofing compound was used in the concrete, reliance for water-tightness being placed on obtaining a concrete of maximum density. As the form progressed, the outer surface of the wall was rubbed with carborundum bricks, while wet. After the wall was completed, its outer surface was given a second, final rubbing-down, from the top down.

The last work done was to place the elastic filler in the V-shaped groove between the base and the wall. The groove was cleaned thoroughly, and was made warm, by means of torches, before the elastic filler, which consists of a preparation, known as *Positive Seal* 24,† was poured, after being first heated to a temperature of 350 to 400° F. The pouring was done uniformly in the whole joint, the height of the filler being gradually raised.

381. *The roof* is made of the same 1 : 2 : 4 mixture of concrete, which was used in the base. The concrete was laid continuously, and as soon as the forms were removed, the surface was rubbed smooth with carborundum stones. A cast-iron manhole with cover is placed where the ladder enters the stand-pipe.

382. *Pipes.* — The outlet pipe is 18 ins. in diameter, and is controlled by a gate-valve, placed in a chamber of reinforced concrete. A gutter is provided around the stand-pipe, to collect any leakage that may occur. The total cost of the stand-pipe amounted to $27,533.64.

* This arrangement was patented by A. D. Whipple, Milwaukee, Wis.

† Manufactured by the Barber Asphalt Co.

TABLE L. — STAND-PIPES OF REINFORCED CONCRETE

Location	Inside diam. in ft.	Height of tank in ft. and in.	Depth of water in ft. and in.	Capacity in gallons	Date of construction	Total cost	References
Little Falls, N. J.	10	43	43	25,260	1899	......	Trans. Am. Soc. C. E., June, 1903, p. 454
Milford, Ohio	14	81	78	93,000	1903	......	Eng. News, Feb., 1904, p. 184
Fort Revere, Mass.	20	50	49	118,000	1903	$ 4,000	Jour. N. E. W. W. A., 1905, p. 33
Attleboro, Mass.	50	102	100	1,500,000	1904	35,000	Jour. N. E. W. W. A., 1906, p. 302
Waltham, Mass.	100	37	35	2,000,000	1906	26,000	Jour. N. E. W. W. A., 1907, p. 325
Bondsville, Mass.	70	20	20	576,000	1908	......	Jour. Assoc. Eng. Soc., June, 1911, p. 399
Empalme, Mex.		90	90	475,000	1908	......	Eng. Rec., Sept., 1909, p. 350; Jan., 1910, p. 137. Eng. News, Dec., 1909, p. 635
New Haven, Conn.	50	25	....	375,000	1908	......	Proc. Conn., C. E., 1908, p. 155. Eng. News, Vol. 59, p. 191
Lenoir, N. C.	....	....	....	500,000	1908	......	
Bridgewater, Mass.	30	78	78	413,000	1909	......	Jour. Assoc. Eng. Soc., June, 1911, p. 399
Manchester, Mass.	50	72	70	1,060,000	1909	30,291	Jour. Assoc. Eng. Soc., June, 1911, pp. 392 and 401
Lisbon Falls, Me.	50	62	60	910,600	1909	19,288	Jour. Assoc. Eng. Soc., June, 1911, p. 398
Westerly, R. I.	40	70	70	650,000	1910	18,722	Trans. Am. Soc. C. E., June, 1911, p. 373. Jour. Assoc. Eng. Soc., June, 1911, p. 402, etc.
Rockland, Mass.	46	104	102	1,300,000	1910	36,300	Jour. Assoc. Eng. Soc., June, 1911, pp. 392 and 396
Cherry Valley, Mass.	40	21 4	20 9	195,000	1910	4,976	Jour. Assoc. Eng. Soc., June, 1911, p. 411
Rochdale, Mass.	40	21 4	20 9	195,000	1910	4,976	Jour. Assoc. Eng. Soc., June, 1911, p. 411
Kensington, Conn.	50	21	20 5	300,000	1910	5,100	Eng. Rec., Feb., 1911, p. 183
Key West, Fla.	78	40	....	1,500,000	1911	24,950	Eng. News, Vol. 65, April 20, 1911, p. 492
Laconia, N. H.	28	46 1	43 6	200,000	1911	6,575	
Brockton, Mass.	160 each	26 6	25	3,760,000 each	1911	82,200	Annual Report of Water Supply Dept., 1911
Western, Mass.	50	38	36	441,000	1911	6,706	
Waverley, Ohio	16	82	80	120,000	1911	4,500	Eng. Rec., July, 1911, p. 137
Ashland, Mass.	40	32 2	31 8	298,000	1911	5,810	Jour. Assoc. Eng. Soc., June, 1911, p. 412
Northbridge, Mass.	25	28	27	90,000	1911	2,899	
Suffern, N. Y.	69	20 6	19 9	559,000	1911	6,500	Jour. N. E. W. W. A., June, 1912, p. 138
Lexington, Mass.	30	104 6	104	550,000	1912	19,900	
Belton, Tex.	24	75	75	254,000	1912	6,000	Eng. & Cont., March, 1913, p. 355
Winchester, Mass.	29	43 6	40 6	200,000	1912	8,000	
Penetanguishene, Canada	50	21	20 5	300,000	1912	......	Eng. & Cont., Jan., 1913, p. 110
Austin, Minn	40	29 8	29	300,000	1912	......	Eng. Rec., March 30, 1912, p. 356
Topsham, Me.	97	47 9	46 3	2,500,000	1913	38,000	
Fulton, N. Y.	40	104 4	100	940,000	1913	24,335	Eng. Rec., Jan., 1914, p. 43. Eng. & Cont., April, 1914, p. 460
San Francisco, Cal.	60	35 10	36 5	750,000	1913	......	Eng. News, Dec., 1913, p. 1204
St. Louis, Mo.	153.5	33	31	4,250,000	1913	51,850	Trans. Am. Soc. C. E., Dec., 1914, p. 1052
Chelmsford, Mass.	40	20	20	188,000	1913	5,180	
West Falmouth, Mass.	30	45	45	238,000	1913	9,800	
Woonsocket, R. I.	79	45	44	1,600,000	1913	23,514	
Sioux City, Ind.	142	33 1	32 1	....	1913	......	Eng. & Cont., Feb. 12, 1913, p. 177
Duxbury, Mass.	40	35	35	328,000	1914	7,115	
Webster, Mass.	46	20	20	249,000	1914	5,260	
Jamestown, R. I.	35	50	49	350,000	1914	10,010	
Halifax, N. S.	160	29	....	3,250,000	1914	56,000	Canadian Engr., March 25, 1915, p. 381

383. *Weather Conditions.* — This stand-pipe was constructed during the worst season of the year for concreting, the wall being built in November. It was with extreme difficulty, due to low temperature, snow, ice, and sleet storms, that the continuity of concreting, which has been decided upon for the wall, was carried out. Nevertheless, the absence of horizontal shear-cracks at or near the base of the completed structure, proves the perfect success of the slip joint

CHAPTER XVIII

CYLINDRICAL TANKS OF WOOD OR STEEL

384. Cylindrical tanks, erected on high ground, or on a tower or trestle, may be used as service-reservoirs for small communities, or to hold a reserve supply of water for fire protection. Many tanks are being put up for the latter purpose, to protect factories and important buildings. When a suitable elevated site can be found near a town, a stand-pipe is often erected, but in flat regions, the elevated tank is preferable to a stand-pipe, as practically its whole capacity is available for fire protection. For this purpose, however, its bottom should be placed at least 80 to 100 ft. above the top of the highest building in the town. The capacities of cylindrical tanks are given in Table LI.

Cylindrical Wooden Tanks *

385. Materials. — The tanks are made of white cedar, cypress, white and red pine, Douglas or Washington fir or air-dried redwood. The lumber must be free from sap, loose or unsound knots, worm holes and shakes; and must be thoroughly air-dried. If made of good material and properly maintained, the tanks will last for 15 to 25 years. For tanks not exceeding 16 ft. in diameter and 16 ft. in depth, the bottom and staves are made of $2\frac{1}{2}$-in. stock, and are dressed on both sides to a thickness of about $2\frac{1}{4}$ ins. For larger tanks 3-in. stock (dressed to about $2\frac{3}{4}$ ins.) should be used.

The groove in the staves for receiving the bottom must be cut in a true line, at a uniform distance from the bottom of the staves, and to a depth of $\frac{5}{8}$ or $\frac{3}{4}$ in., according to the thickness of the staves. The edges of each bottom plank must be bored with holes, not over 3 ft. apart, for $\frac{5}{8}$-in. maple dowels.

386. *Hoops.* — The strength of the tank depends chiefly on that of its hoops. Flat hoops, especially if made of steel, rust from the side bearing on the staves. Serious accidents have resulted from this cause, as the corrosion on the back of the hoops cannot be observed. Gal-

* The information given about wooden tanks is taken largely from the very complete specification for "Gravity Tanks and Towers," prepared by The Inspection Department of the Associated Factory Mutual Fire Insurance Companies, Boston, Mass. Figs. 199 to 203 incl. are reproduced from these specifications.

vanizing or painting is not a sure remedy for this trouble, as a spot where the protective coating has been abraded or knocked off may cause the failure of the hoop. Round hoops, which expose for a given area of cross-section the least surface to corrosion, should always be used for wooden tanks. They are, also, less liable than flat hoops to burst when

TABLE LI. — CAPACITY OF CYLINDRICAL TANKS IN GALLONS PER LINEAL FOOT

Diameter in ft.	Capacity in gals.	Diameter in ft.	Capacity in gals.	Diameter in ft.	Capacity in gals.
1	5.9	35	7,197	69	27,972
2	23.5	36	7,614	70	28,788
3	52.9	37	8,043	71	29,617
4	94	38	8,484	72	30,457
5	146.9	39	8,936	73	31,309
6	211.5	40	9,400	74	32,173
7	287.9	41	9,876	75	33,048
8	376	42	10,364	76	33,935
9	475.9	43	10,863	77	34,834
10	587.5	44	11,374	78	35,745
11	711	45	11,897	79	36,667
12	846	46	12,432	80	37,601
13	993	47	12,978	81	38,547
14	1152	48	13,536	82	39,505
15	1322	49	14,106	83	40,474
16	1504	50	14,688	84	41,455
17	1698	51	15,281	85	42,488
18	1904	52	15,887	86	43,453
19	2121	53	16,503	87	44,469
20	2350	54	17,132	88	45,498
21	2591	55	17,772	89	46,537
22	2844	56	18,425	90	47,589
23	3108	57	19,089	91	48,653
24	3384	58	19,764	92	49,727
25	3672	59	20,452	93	50,815
26	3972	60	21,151	94	51,913
27	4283	61	21,862	95	53,024
28	4606	62	22,584	96	54,146
29	4941	63	23,319	97	55,280
30	5288	64	24,065	98	56,425
31	5646	65	24,823	99	57,583
32	6016	66	25,592	100	58,752
33	6398	67	26,374		
34	6792	68	27,167		

the staves swell, as they simply indent the wood, in such a case, and practically their whole surface can be inspected, and painted when required.

The hoops should be made of wrought iron or mild steel having, respectively, a tensile strength of not less than 50,000 lbs. per sq. in. and 55,000 to 60,000 lbs. per sq. in. There must be no welds in the hoops. When more than one length of iron or steel is required, malleable iron lugs (Fig. 215), as strong as the hoop, should be used. Cast iron lugs should not be used, as they are liable to crack. The hoops must be so placed that the lugs will not come in the same vertical plane.

The ends of the hoops should not be upset, as the metal is likely to be burned, if the work is not carefully done. When the screw threads are cut directly on the ends of a hoop, the unthreaded portion may be corroded to a depth equal to that of the screw threads, without weakening the hoops.

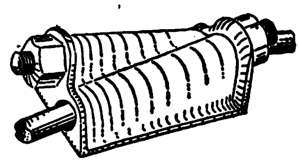


FIG. 215. — Lug for Hoops.

The size and spacing of the hoop must be such that the metal will not be stressed more than 12,500 lbs. per sq. in., computed for the area at the root of the thread. The diameter of the hoops should never be less than $\frac{3}{4}$ in. The top hoop should be placed within 2 ins. of the top of the staves, and no space between the hoops should exceed 21 ins. Table LII shows the spacing adopted by the Associated Factory Mutual Fire Insurance Companies.

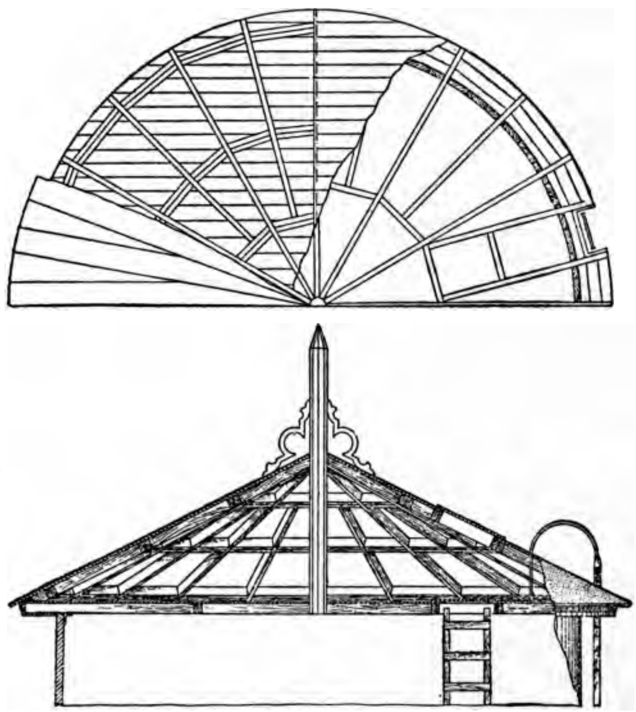


FIG. 216. — Double-cover Roof.

387. Painting. — Tanks should be painted on the outside and inside. The hoops should first be painted with red lead, zinc oxide, and linseed oil, and should then be given a second coat of paint, consisting of some durable oil or asphaltum paint, which should be applied after the tank has been erected. The inside of the tank should be repainted about once every two years or oftener, if the paint shows signs of peeling. The outside should be repainted at intervals of about five years.

388. Roof. — Tanks that are placed out-of-doors should be covered with a double roof, consisting of a tight flat cover, made of matched boards, supported by joists, over which a conical roof is placed. For large tanks, the conical roof must be supported by rafters extending from the top of the tank to the peak of the roof (Fig. 216).

TABLE LII. — STANDARD WOODEN TANKS

Capacity in gals.....	10,000		15,000		20,000		25,000		30,000		40,000		50,000	
Outer diameter..	13′ 4″		14′ 6″		15′ 6″		17′ 6″		18′ 0″		19′ 6″		22′ 6″	
Stave, length.....	12′ 0″		14′ 0″		16′ 0″		16′ 0″		18′ 0″		20′ 0″		20′ 0″	
Thickness	2¼″		2¼″		2¼″		2¾		2¾″		2¾″		2¾″	
	Size and spacing of bands, center to center													
Number of band from top	Size, in.	Space, ins.	Size, in.	Space, ins.	Size, in.	Space, ins.	Size, in.	Spaces, ins.	Size, in.	Space, ins.	Size, in.	Space, ins.	Size, in.	Space, ins.
1	¾	0	¾	0	¾	0	¾	0	¾	0	¾	0	⅞	0
2	¾	21	¾	21	¾	21	¾	21	¾	21	¾	21	⅞	21
3	¾	21	¾	18	¾	18	¾	18	¾	18	¾	20	⅞	21
4	¾	18	¾	18	¾	18	¾	18	¾	18	⅞	19	⅞	18
5	¾	15	¾	15	¾	15	⅞	15	⅞	15	⅞	16	1	18
6	¾	15	¾	15	⅞	15	⅞	15	⅞	15	⅞	14	1	15
7	¾	12	¾	12	⅞	15	⅞	15	⅞	12	⅞	12	1	15
8	¾	12	¾	12	⅞	12	⅞	12	⅞	12	⅞	11	1	12
9	¾	10	¾	11	⅞	12	⅞	12	⅞	12	⅞	10	1	12
10	¾	8	¾	10	⅞	11	⅞	11	⅞	10	⅞	9	1	10
11	¾	6	¾	9	⅞	10	⅞	10	⅞	10	⅞	9	1	10
12	....	....	¾	8	⅞	9	⅞	9	⅞	9	⅞	8	1	6
13	....	....	¾	7	⅞	9	⅞	9	⅞	9	⅞	8	1	6
14	....	....	¾	6	⅞	8	⅞	8	⅞	8	1	8	1	8
15	....	....	....	....	⅞	7	⅞	7	⅞	8	1	8	1	8
16	....	....	....	....	⅞	6	⅞	6	⅞	7	1	8	1	8
17	....	....	....	....	....	....	....	....	⅞	7	1	8	1	7
18	....	....	....	....	....	....	....	....	⅞	7	1	8	1	7
19	....	....	....	....	....	....	....	....	⅞	6	1	8	1	7
20	....	....	....	....	....	....	....	....	⅞	6	1	7	1	6
21	....	....	....	....	....	....	....	....	....	....	1	7	1	6
22	....	....	....	....	....	....	....	....	....	....	1	7	1	5
23	....	....	....	....	....	....	....	....	....	....	1	6	1	3
24	....	....	....	....	....	....	....	....	....	....	1	3	....	.....

Notes. The first band is placed 2 ins. below the top of the tank, and the last band is placed opposite the center of the flooring. The bottom of the stave projects about 3½ ins. below the bottom of the flooring. The thickness given is for the dressed stave. The bottom is made the same thickness.

The conical roof should be covered with galvanized sheet iron, or a good composition roofing which is not readily ignitible. The joint between this roof and the flat cover should be made tight, in order to keep out the wind, and to maintain a dead air space between the two covers. Hatches must be provided both in the conical and the flat covers, to give easy access to the interior of the tank.

389. Supports. — Large wooden tanks are either placed on a masonry foundation or on twelve or more timber posts, usually 12 by 12 ins. in size. Each post is set on a masonry pier. The weight of the tank must be supported entirely from its bottom, and there should be a clear space of at least one inch between the bottom or outside of the staves and the balcony, which is usually provided for tanks 30 ft. or more in height.

390. Pipes. — The filling pipe should be at least $1\frac{1}{2}$ ins. in diameter. When the tank is exposed to the weather, this pipe should be carried up inside of a frost-proof casing, and should extend through the tank bottom, to discharge at the top of the tank, above the water level. The portion of the pipe inside of the tank should be of brass.

The discharge pipe is made of cast iron or wrought iron pipe, flanged or coupled. Copper gaskets should be placed between the flanges. This pipe usually leaves the tank bottom at its center, and is supported on an underground elbow having a foot-piece, resting on a concrete foundation. The elbow should be connected, preferably by flanges, to the underground pipe. If a bell-and-spigot connection is used, the joint should be strapped. A check-valve should be placed in the underground pipe, in a suitable pit, so as to make it accessible for inspection and repairs. On each side of this check-valve, a gate-valve should be placed, in order to make it possible to repair the check-valve without emptying the tank.

A suitable outlet-pipe, provided with a gate, must be placed in the tank bottom to drain the bottom of the tank below the upper end of the discharge pipe which projects for a few inches into the tank. It may be connected by a nipple with a gate-valve to the discharge pipe, so as to drain the tank and the riser.

When a tank is supported on a tower, 30 ft. or more in height, an expansion joint must be provided in the discharge pipe. The best arrangement for this purpose is made by means of a stuffing-box (Fig. 217). The stuffing-box casting should project 4 ins. within the tank, to provide a basin for sediment. The riser pipe must be fitted so that its upper end is about 5 ins. below the top of the expansion joint.

An overflow pipe, 2 ins. or more in diameter, according to the size of the tank, must be provided. Its top should be placed 3 ins. below the top of the staves, and the pipe should extend through the bottom or side of the tank. In the latter case, it should project beyond the balcony.

391. Heating arrangements must be made in cold localities to keep the water from freezing. The water may be heated by circulating it through a heater, at the base of the riser, which may consist of a

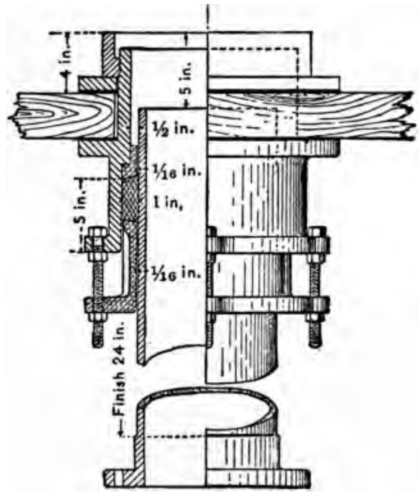


FIG. 217. — Expansion Joint.

steam-coil, enclosed by a water jacket, connected at one end to the base of the riser, below the frost line, and at the other end to the tank, through a pipe extending up alongside the riser, within the frost-proof casing, through the bottom, and discharging at about the middle of the pipe. The condensed steam should be drained from the steam coil by connecting it to a steam-trap, which should be placed close to the heater, where it can be readily taken care of.

Another method of heating the water consists in fastening a steam coil securely, 6 ins. above the bottom of the tank, and connecting it to a boiler, by a pipe run close to the discharge pipe, inside the frost-proof boxing. All pipes within the tank should be of brass. The steam pipe should extend below the frost line, and the condensed steam should be drained to a trap.

In localities where heating is only needed for short periods during the winter, steam may be blown directly into the water in the tank, through a 1-in. pipe from a boiler. The pipe should be carried up, in the frost boxing, through a hole in the bottom of the tank to a point above the water surface, and then turned down, at least 3 ft., into the water. The temperature of the water in the tank should not be allowed to fall below the freezing point or to rise above 70° F.

392. Frost-Proofing for Pipes. — The various pipes required for a water tank should be protected by a suitable boxing against frost, but this protection should not be depended upon without some steam heat being added. In tanks that are used solely for fire protection, there is no circulation in the water, and more protection against frost is required in such tanks than in those used for water supply.

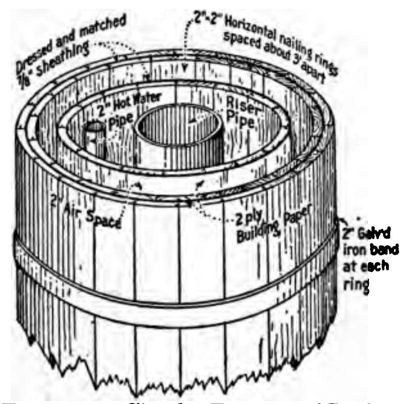


FIG. 218. — Circular Frost-proof Boxing.

The standard frost-proof boxing (Figs. 218 and 219) is made of wood. If antiseptically treated lumber is used, the boxing will be more durable. A tight joint must be made between the boxing and the bottom of the tank, and the lower end of the boxing should extend about a foot above ground. The woodwork must be well painted. Sheet lead or tarred paper should be placed between the bottom of the boxing and the pit, to avoid the absorption of moisture.

The upper part of the boxing must be made so that access can be gained to the expansion-joint, without destroying the boxing. For tanks in northern Canada, the boxing must be made four-ply, with two air spaces. For the New England States and places having similar climate, the boxing is made three-ply, with two air spaces.

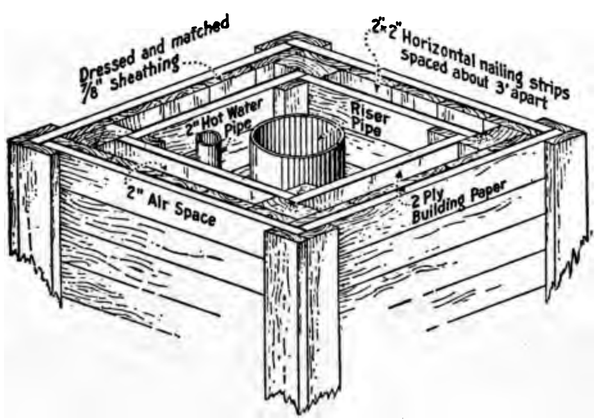


FIG. 219. — Square Frost-proof Boxing.

393. Gauges. — A mercury gauge, or some other device for indicating positively the level of the water surface, should be provided.

394. Ladders. — A steel ladder, extending from the platform to the top of the tank, should be placed on the outside of the tank so as to give easy access to the hatch on the roof.

Steel Tanks

395. General Descriptions. — During the past twenty years, the erection of elevated steel tanks, supported by columns, has progressed very rapidly. When properly designed, they can resist safely any wind pressure to which they may be subjected. They are easily inspected, cleaned, and painted, and should have, therefore, a long life.

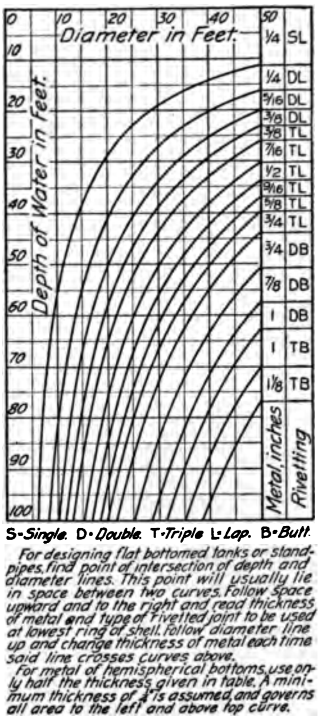


Fig. 220.

When steel tanks were first introduced, about 40 or 50 years ago, their bottoms were made flat, as is customary with wooden tanks. This plan is, however, very costly. Some tanks were made with conical bottoms, but this plan has no advantages. Finally, in 1894, the bottoms were made spherical, and this plan is now generally adopted, as it gives great strength to the bottom.

Tanks of less than 40,000 gals. capacity can be made more cheaply of wood than of steel, but for larger tanks the latter material should be used, not only because it is cheaper, under ordinary market conditions, but, also, because it will last longer. Steel tanks are easily supported, and for capacities up to 150,000 gals., require only four columns, if they are properly braced.

A steel tank having a capacity of 1,200,000 gals., elevated 220 ft. above the ground, was erected for the waterworks of Louisville, Kentucky. It is the largest elevated steel tank, but there is no reason why its dimensions should not be exceeded. A great many elevated steel tanks with capacities of 50,000 to 300,000 gals. have been erected and have proved to be satisfactory. Table LIII gives the sizes and elevations of some of these tanks.

396. Standard Plans. — The leading manufacturers of steel tanks have standardized their plans. Fig. 220 gives a diagram for designing steel tanks and stand-pipes, which has been prepared by the Pittsburgh-

PLATE VIII.

Steel Water Tank at St. Thomas, Ont. — Capacity 600,000 gallons.
(Built by Pittsburgh-Des Moines Steel Co.)

Des Moines Steel Co.* The standard dimensions of the steel tanks manufactured by this company are given in the following table:

TABLE LIII. — STEEL TANKS FOR WATER WORKS

Locality	Capacity in gals.	Height in ft.
Louisville, Ky.*	1,200,000	220
Stratford, Ont.*	600,000	155
St. Thomas, Ont.†	600,000	131
Lakewood, Ohio †	560,000	70
Appleton, Wis.*	500,000	140
Thompsonville, Conn.†	500,000	104
Ely, Minn.†	300,000	189
Fort Missoula, Mon.†	300,000	155
Fort Sam Houston, Texas †	300,000	103
Sterling, Colo.†	253,000	103
Pearl Harbor, Hawaii †	250,000	144
Norton, Mass.*	200,000	175
Huron, So. Dak.†	200,000	130
Emporia, Va.†	200,000	120
Howell, Ind.*	200,000	109
Vineland, N. J.*	185,000	120
Chicago, Ill.*	180,000	145
Camden, N. J.*	150,000	242
Centreville, Iowa †	150,000	125
Fairfield, Iowa †	150,000	124
Spring Valley, Ill.*	150,000	115
Winnipeg, Canada *	125,000	135
Monmouth, Ill.*	115,000	127
Boston, Mass.*	100,000	239
Little Rock, Ark.*	100,000	159
College Hill, Ohio *	100,000	154
Columbus, Ohio *	100,000	150
Redford, Mich.†	100,000	131
Exeter, Calif.†	100,000	128
Julesburg, Colo.†	100,000	114
North Rutland, Vt.*	70,000	220
Rupert, Idaho †	50,000	130

* Constructed by the Chicago Bridge & Iron Works, Chicago, Ill.
† Constructed by the Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

TABLE LIV. — STANDARD STEEL TANKS
(Pittsburgh-Des Moines Steel Co.)

Rated capacity, U. S. gallons	Diameter D	Distance h	Cylinder C	Rated capacity, U. S. gallons	Diameter D	Distance h	Cylinder C
	Ft. Ins.	Ft. Ins.	Ft. Ins.		Ft. Ins.	Ft. Ins.	Ft. Ins.
10,000	11 0	4 0	10 11	70,000	21 0	7 7	21 10
15,000	13 0	6 0	10 11	75,000	21 0	8 0	22 6
20,000	15 0	5 5	10 9	80,000	21 0	8 11	24 2
25,000	15 0	5 4	14 7	90,000	21 0	8 0	28 4
30,000	15 0	5 3	18 5	100,000	24 0	8 6	22 6
35,000	17 0	6 4	15 7	125,000	24 0	8 4	30 0
40,000	17 0	6 8	18 5	150,000	28 0	10 5	24 2
45,000	19 0	6 5	15 11	200,000	28 0	10 5	35 0
50,000	19 0	8 0	17 7	250,000	34 0	14 8	25 10
60,000	19 0	7 3	22 6	300,000	34 0	14 3	33 4

* Formerly the Des Moines Bridge & Iron Co.

Fig. 221 shows the standard plans adopted for the tanks given in the above table.

397. Stresses in steel tanks can be computed by a few simple formulas, which are given below, some of which are, also, applicable to stand-pipes. The notation adopted is as follows:

- h = depth of water in feet;
- d = diameter of tank in feet;
- r = radius of tank in feet;
- y = $12\,r$ = radius of tank in inches;
- t = thickness of side plates in inches;
- p = hydrostatic pressure in pounds per square inch;
- S = stress per vertical inch of tank;
- s = unit stress in a vertical section of tank, in pounds per square inch;
- S' = stress per horizontal inch of tank;
- s' = unit stress in a horizontal section of tank in pounds per square inch;
- S'' = stress per lineal inch of circumference, due to wind;
- s'' = unit stress in pounds per square inch along the circumference, due to wind;
- T_1 = radial stress per square inch in a hemispherical bottom;
- T_1' = radial stress per square inch in a segmental bottom;
- w = weight of a cubic foot of water;
- W = weight of tank in pounds;
- M = bending moments in inch-pounds at a distance of h below the top;
- I = moment of inertia.

According to the well-known pipe formula, we have

$$S = \frac{whd}{12 \times 2} = 2.6\,hd. \qquad (88)$$

Dividing by the thickness t of the plates, we obtain the unit stress per square inch:

$$s = \frac{2.6\,hd}{t}. \qquad (89)$$

The stress per horizontal inch of a tank, caused by the weight of the tank is

$$S' = \frac{W}{12\,\pi d} = \frac{0.026\,W}{d}, \qquad (90)$$

from which we obtain the unit stress per square inch:

$$s' = \frac{0.026\,W}{dt}.$$

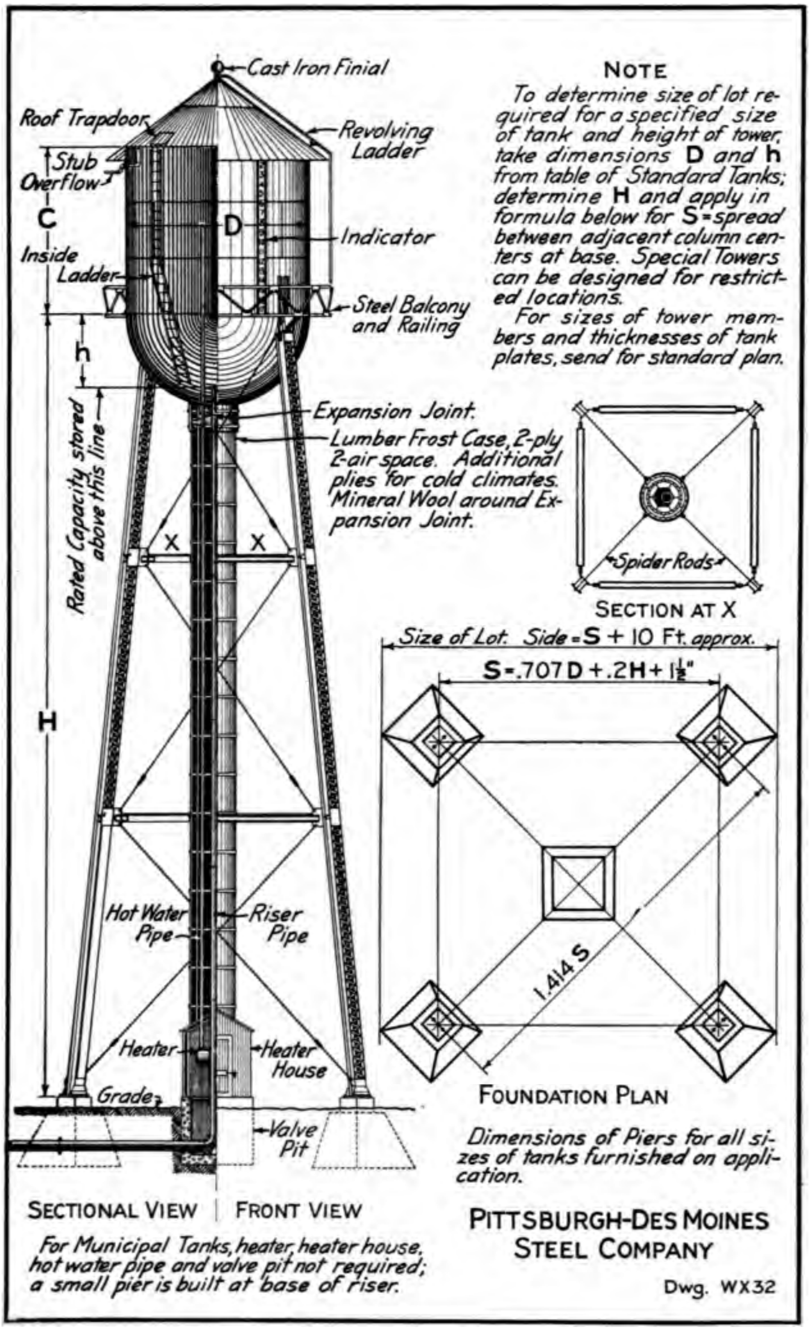


Fig. 221. — Standard Plan for Elevated Steel Tank.

The stresses caused on a stand-pipe by wind pressure are determined as follows:

The wind pressure is assumed, in ordinary cases, at 30 lbs. per sq. ft., acting on two-thirds of the surface, or at 20 lbs. per sq. ft. on the entire surface. In exposed positions, the wind pressure should be taken as 40–50 lbs. per sq. ft., acting on two-thirds of the surface, or at about 30 lbs. on the entire surface.

Assuming 20 lbs. per sq. ft. as the wind pressure on the whole surface, we obtain

$$M = 20 \times dh \times h \times \tfrac{12}{2} = 120\,dh^2. \tag{92}$$

From the above equation we find the unit stress in the extreme fiber of the shell to be

$$s'' = \frac{My}{I} = 1.06\,h^2\,(td) \text{ approximately.} \tag{93}$$

The method of computing wind-stresses in elevated tanks is explained on p. 393.

398. *Spherical Bottom.** — If the bottom of the tank is a hemisphere, the radial stress per square inch will be one-half of the stress in a cylinder of the same radius, subjected to the same internal pressure. We have, therefore,

$$T_1 = \frac{2.6\,hd}{2t} = \frac{2.6\,hr}{t}. \tag{94}$$

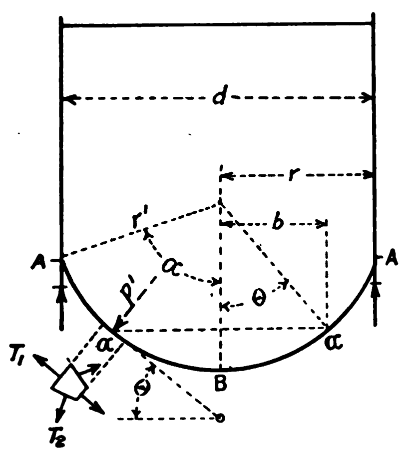


FIG. 222. — Segmental Bottom for Steel Tank.

In a segmental bottom (Fig. 222), let W' represent the weight of water to be supported by the tank bottom at the level a–a, and let T_1' be the unit stress in the radial joint. We shall have

$$T_1' = \frac{W' \times \csc\theta}{2 \times 12\,\pi bt} = \frac{W' \times \csc^2\theta}{24\,\pi r_1 t}. \tag{95}$$

Substituting in the above equation for W' its value $62.5\,h\pi b^2 = 62.5\,h\pi r_1^2 \sin^2\theta$, we obtain

$$T_1' = \frac{62.5\,hr_1}{24\,t} = \frac{2.6\,hr_1}{t}. \tag{96}$$

When $r_1 = r$ the above equation becomes equation (94) for a hemispherical bottom.

* "Structural Engineers' Handbook," by Milo S. Ketchum, C. E., p. 366.

399. The towers for steel tanks have usually four columns, inclined where possible on a batter of not less than 1 in 12. The inward thrust exerted by the posts against the tank is opposed by a circular girder, not less than 24 ins. in width, placed in such a position with regard to the connection that the bending moment produced in the post is reduced to

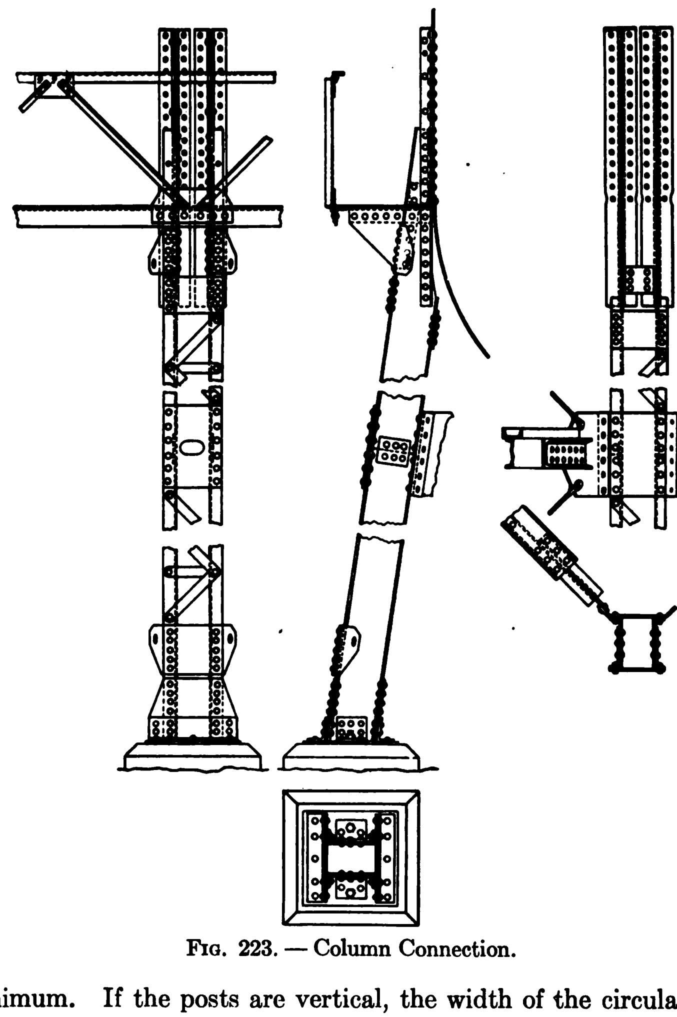


FIG. 223. — Column Connection.

a minimum. If the posts are vertical, the width of the circular girder may be reduced to 18 ins. Each column must be securely anchored to the foundation, so as to be able to withstand the maximum uplifting stresses caused by the wind acting on an empty tank. The columns are supported, at intervals of 20 to 40 ft., by lateral bracing, which, also, forms wind-bracing. The tower is designed according to the usual methods for structural work, the effect of wind pressure being computed as explained on page 393.

In order to avoid eccentric loading in the posts and local stresses in the tank plates, the connection between the posts and the tank must be made in such a manner that the center of gravity of the column sections shall intersect the tank at the center of the girder connection. Fig. 223 shows a good design for this connection.

The cap and base plates should be made of steel. The latter should have sufficient thickness and size to distribute the weight on the foundation pier without causing excessive pressure. The pressure on the bear-

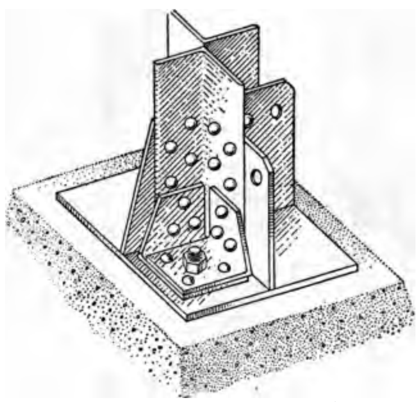


Fig. 224. — Base-plate for Angle Post.

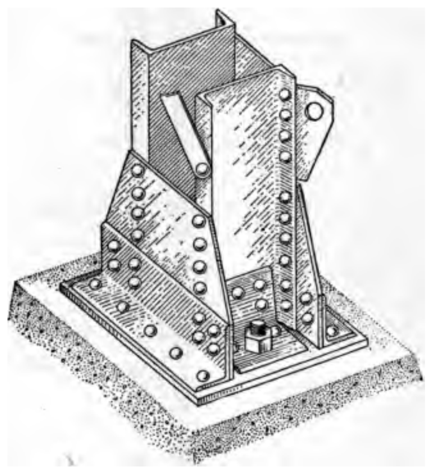


Fig. 225. — Base-plate for Channel Post.

ing plate should not exceed the following safe limits, expressed in pounds per square inch:

Portland cement concrete	400
Sandstone (first class)	400
Limestone (first class)	500
Granite (first class)	600
Hard brick with Portland cement mortar	200

Figs. 224 and 225 show approved designs for the base of the columns.

Each column should be securely anchored to its foundation by anchor bolts of wrought iron or mild steel bearing against anchor plates, not less than ½ in. thick under the nut, and set in the foundation piers several inches above the bottom. These bolts must be strong enough to resist safely the stresses caused by wind pressure and, also, the shearing forces on the column footing.

The wind bracing should be made either of wrought iron or steel rods, arranged for tightening, or built up of riveted structure members.

400. The foundation piers should be pyramidal in shape, with a height not less than the mean width. When the columns are inclined, the piers must be so designed that the resultant of the vertical and horizontal forces, due to direct loading, shall pass through the center of

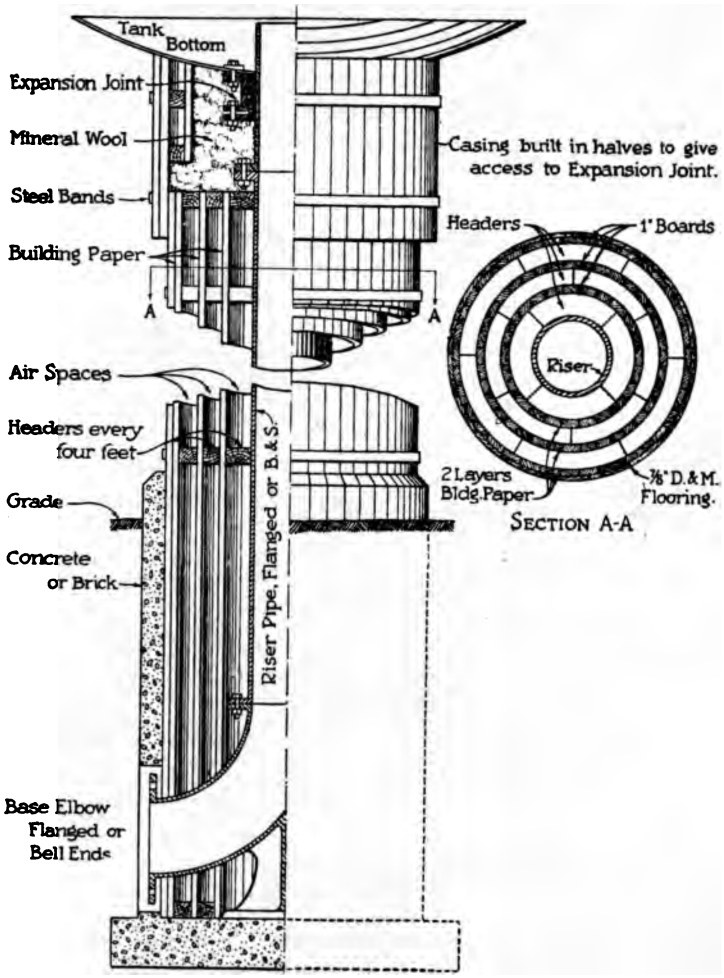


FIG. 226. — **Frost Case with 3-ply Building Paper and 3 Air Spaces. (Pittsburgh, Des Moines Steel Co.)**

gravity of the pier. The piers should be founded below the frost level, and the tops must be about 12 ins. above ground. The weight of the pier when buried for at least two-thirds of its height must be equivalent to the calculated net uplift, otherwise the weight should be $1\frac{1}{2}$ times this amount.

401. Inlet-pipe. — In the first designs for steel tanks, the inlet-pipes, called risers, consisted of small cast iron pipes which were protected against frost, in cold localities, by wooden casings. For modern steel tanks for water supply, the riser consists usually of a large riveted steel pipe, the tank having a full diameter opening at the connection with the riser. The diameters of the risers vary from 3 ft. in the South-

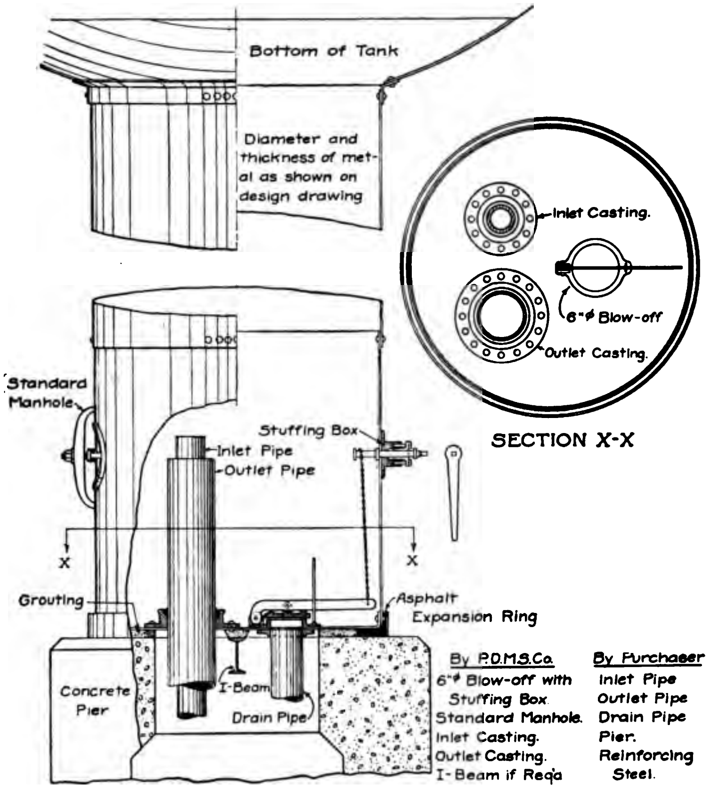


FIG. 227. — Water-heater and Circulating System.
(Pittsburgh, Des Moines Steel Co.)

ern States to 6 ft. in the Northern States and Canada, where the winters are severe. In these large risers, an insulating layer of ice forms next to the steel, but a free passage remains for the water through the center of the pipe. No frost casing is, therefore, needed in such cases. The 4-ft. riser of the 1,200,000 steel tank at Louisville, Ky., has no frost protection.

For small tanks, however, the risers have smaller diameters than 4 ft., and this necessitates in cold regions a frost protection. Such a protection is, also, required in cold places for tanks used for fire protection in which there is no circulation, except when water is drawn to extinguish fires. A good plan of making a frost casing is shown in Fig. 226. According to circumstances, the casing can be made 2-, 3-, or 4-ply. Mineral wool is used around the expansion-joint at the top of the riser.

A flanged head forms the bottom of the riser and rests upon a concrete pier. Flanges placed in this head admit the inlet- and outlet-pipes which extend upwards a few feet into the riser, so that all water is drawn off well above the level of the mud or other impurities which may have accumulated in the bottom. This deposit must be removed periodically

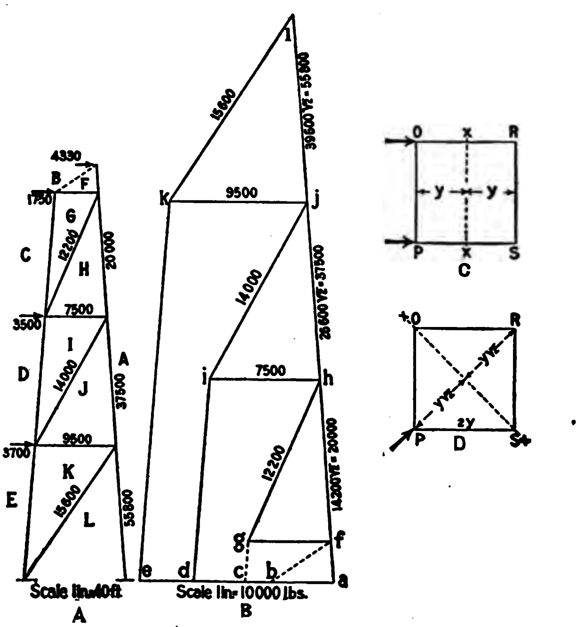


FIG. 228. — Diagram for Calculating Wind Stresses in a Four-post Tower.

by means of a blow-off valve, without interrupting the service of the tank. A suitable manhole is provided at the bottom of the riser. Fig. 227 shows a standard plan for the bottom of the inlet-pipe. Small tanks which are used for fire protection in connection with automatic sprinkler systems may require some heating systems similar to those described on p. 382, in addition to a frost casing.

402. General Requirements. — The arrangements as regards roof pipe, connecting balcony, ladders, and painting are about the same for steel tanks as those made of wood. The roof is always made conical, and must have a hatch, to give admittance to the tank.

403. Wind Stresses in Elevated Tanks (Fig. 228). — The following method of calculating wind stresses in a four-post tower has been adopted by the Inspection Department of the Associated Factory Mutual Fire Insurance Companies of Boston, Mass.

"The problem involves a simple application of graphic statics, and depends upon the proposition that a force is fully determined when its magnitude, direction, and point of application are known. Such a force may be represented by a line, and the stress diagram in its simplest form represents the force as sides of a polygon, taken in order. The closing side, in reverse order, is the resultant in magnitude and direction. The diagram shown on page 393 is a typical one, and the method of computation is as follows:

"*A* represents one bent of a 100-ft. tower, supporting a 40,000-gal. tank. The loadings beginning at the top of the diagram are 4330 lbs., applied at center of gravity of the projected surface of tank and roof; 1750 lbs., which is equal to 100 lbs. per ft. for one-half the height of the top stage; 3500 lbs., which is equal to 100 lbs. per ft. for one-half the heights of the top and middle stages; and 3700 lbs., which is equal to 100 lbs. per ft. for one-half the heights of middle and bottom stages. One-half the wind load is used because one bent is being considered.

"*B* shows the stress diagram and, as it is drawn to scale, the force resulting may be measured directly from it.

"*C* is a plan-view of *a*, and shows the wind blowing in the plane of one bent. This being the case, the axis of rotation will be through *XX* and posts *R* and *S* will take compression, while posts *O* and *P* will take tension. The amount of the compression in post *O* has been found from the diagram to be 39,600 lbs. The uplift or tension in the anchor bolt at the foot of post *P* will be a like amount. Suppose, however, we consider the wind blowing in some other direction, and determine which is the worst case for wind loading.

"*D* shows the same plan-view, with the wind blowing in a diagonal direction, as indicated by arrow. The axis of rotation will be through *XX*, and this being the case, post *R* will take all the compression. The relative amounts of the compression in these two axes are shown to be as follows:

"In *C* let *M* represent the total overturning moment, then the compression in *R* and *S* is equal to $\frac{M}{4y}$, while in *D* the compression in *R* equals

$$\frac{M}{2y\sqrt{2}} = \frac{M\sqrt{2}}{4y}. \tag{97}$$

"In other words, the compression in *R* in *D* is the same as the compression in either *R* or *S* in *C*, times $\sqrt{2}$; therefore, the maximum compression due to the load may be taken as the amount scaled from the stress diagram multiplied by $\sqrt{2}$.

"Similarly, the maximum tension in any set of anchor bolts will be equal to the maximum compression in the leeward post in *C*, less one-fourth the total weight of the structure and tank.

"The stresses in the diagonal rods and struts are considered to be the maximum when the wind is blowing as in *C*, and, therefore, may be scaled directly from the diagram."

CHAPTER XIX

FIRE PROTECTION

404. General Considerations. — One of the principal objects for which water works are constructed is to give a community ample protection against fire. The total quantity of water used for this purpose is only a very small percentage of the total consumption of the community; but, when needed, it has to be delivered quickly and frequently taxes the utmost capacity of the works. In many cases, the fire department of a town or city has been unable to cope with a great fire, either because the water works were badly designed or insufficient for fire protection.

In order to control such a fire, enough fire streams must be available, with sufficient pressure to reach the roofs of buildings of ordinary heights, which in most towns will not have more than four stories, and, also, to pass through the flames to the heart of the fire. Small fire streams are of service for fires of isolated buildings and to extinguish fire brands that may fall on roofs, but they are utterly useless for extinguishing great fires, as the water they discharge is evaporated, without reaching the seat of the fire. In such cases, powerful fire streams should be used, and there should be enough of them available, according to circumstances. In the residential or suburban districts of a city, two to four streams may suffice, but in the built-up parts of the city, where property of great value is stored, twenty-five to fifty good fire streams may be required.

The efficiency of the water works system for extinguishing fires will depend upon the quantity of water that can be discharged into a fire, and this is limited by the size of the mains in the street, the number of hydrants within a reasonable distance from a fire, and the pressure that can be maintained at the hydrants. If unlimited means were at the disposal of the engineer, it would be comparatively easy to make ample provision for protection against fire; but, in many cases, the money available for the purpose is limited, and the officials who have to authorize its disbursements do not always realize the necessity of having plenty of hydrants, with good pressure, in the important parts of the town. The prevailing custom of paying private water companies which furnish a town or city protection against fire a fixed annual rental per hydrant inclines the authorities of a community to try to limit the number of hydrants as much as possible.

405. Number and Size of Fire Streams. — In small communities, or in the residential or suburban districts of a city, two to four fire streams, each delivering about 200 gals. per min. under a pressure of 30 to 45 lbs. at the nozzle, according to the size of nozzle used, should be available for any fire that may occur. In the built-up parts of a city, each fire stream should discharge not less than 250 to 300 gals., under a pressure of about 45 lbs. at the nozzle. In case of great fires, two lines of hose can be siamesed into one, and with this arrangement powerful fire engines are able to deliver streams of over 1000 gals. per min.

In order to flood high buildings which are on fire, most cities having a population of 200,000 or more have one or more fire towers on which several fire streams can be joined into one large nozzle, $1\frac{1}{2}$ to 2 ins. in diameter, capable of discharging 500 to 1000 gals. per min.

The fire risk varies in different communities of the same size, according to the closeness, height, and character of the buildings, and according to the business carried on. There is, however, not much difference in the protection against fire required in ordinary American cities, and the number of fire streams that should be available depends in such cases mainly upon the population. J. Herbert Shedd appears to have been the first engineer to devise a formula for calculating the number of fire streams that should be provided in a community of a given size. He based his rule upon his own experience, and upon the recommendations that had been made by committees in various New England towns. Mr. Shedd assumed the average discharge of each fire stream at 200 gals. per min. J. T. Fanning made also an estimate of the number of fire streams that communities of different sizes should have, assuming an average discharge of 281 gals. per fire stream. John R. Freeman and Emil Kuichling have made similar estimates, based upon an average delivery of 250 gals. per fire stream. For comparison we give in the following table the number of fire streams required in ordinary American cities and towns of different sizes for proper fire protection, according to the four engineers named above.

Mr. Shedd's formula for calculating the number of fire streams required, according to the size of a community, is

$$\text{Number of fire streams} = 0.005\sqrt{\text{Population}}. \qquad (98)$$

Mr. Kuichling's formula is

$$\text{Number of fire streams} = 2.8\sqrt{X}, \qquad (99)$$

where X = number of thousand inhabitants.

TABLE LV.—NUMBER OF SIMULTANEOUS FIRE STREAMS REQUIRED IN AMERICAN CITIES AND TOWNS

Population	Shedd,* 1 fire stream = 200 gals. per min.	Fanning,† 1 fire stream = 281 gals. per min.	Freeman,‡ 1 fire stream = 250 gals. per min.	Kuichling,§ 1 fire stream = 250 gals. per min.
1,000	....	....	2 to 3	3
4,000	....	7	....	6
5,000	5	8	4 to 8	6
10,000	7	9	6 to 12	9
20,000	10	11	8 to 15	12
40,000	14	13	12 to 18	18
50,000	....	14	....	20
60,000	17	15	15 to 22	22
100,000	22	18	20 to 30	23
150,000	....	23	....	34
180,000	39	....	....	38
200,000	....	27	30 to 50	40
250,000	....	30	....	44
300,000	....	34	....	48

* Jour. New England Water Works Assoc., Vol. III, p. 113.
† Proc. Amer. Water Works Assoc., May, 1892, p. 75.
‡ Jour. New England Water Works Assoc., Vol. VII, p. 55.
§ Trans. Am. Soc. C. E., XXXVIII, p. 15.

In estimating the quantity of water required for extinguishing fires, an allowance should be made for probable losses from broken connections and hydrants left open. Including these losses, the National Board of Fire Underwriters recommend the following fire flows, which are based upon the formula:

$$G = 1020 \sqrt{P} (1 - 0.01 \sqrt{P})$$

where G = gallons per minute and P = population in thousands.

TABLE LVa.—FIRE-FLOW REQUIRED BY NATIONAL BOARD OF FIRE UNDERWRITERS * (1917)

Population	Required fire-flow, gallons per minute for average city	Population	Required fire-flow, gallons per minute for average city
1,000	1000	28,000	5,000
2,000	1500	40,000	6,000
4,000	2000	60,000	7,000
6,000	2500	80,000	8,000
10,000	3000	100,000	9,000
13,000	3500	125,000	10,000
17,000	4000	150,000	11,000
22,000	4500	200,000	12,000

Over 200,000 population, 12,000 gallons a minute, with 2000 to 8000 gallons additional for a second fire.

* Pamphlet issued by the National Board of Fire Underwriters, on "Standard Schedule for Grading Cities and Towns of the United States with Reference to their Fire Defenses and Physical Conditions," New York, 1917.

Note. Residential Districts.—The required fire-flow depends upon the character and congestion of the buildings. Sections where buildings are small and of low height, and with about ⅓ the lots in a block built upon, require not less than 500 gallons per minute. With larger or higher buildings, up to 1000 gallons are required, and where the district is closely built, or buildings approach the dimension of hotels or high value residences, 1500 to 3000 gallons are required, with up to 6000 gallons in densely built section of 3-story buildings.

It is difficult to obtain accurate data from fire departments of the number of fire streams that were playing simultaneously at a fire, the total quantity of water used, etc. Some idea of what may be required in great fires can be obtained from the following data given by John R. Freeman in the Journal of the New England Water Works Assoc., Vol. VII, p. 60, and by the National Board of Fire Underwriters.

TABLE LVI. — SOME GREAT FIRES

Locality	Date	Maximum number of fire streams at one time	Maximum rate of draft for fire above the ordinary draft	Total draft for the fire, until it was practically extinguished
			Gals. per min.	Gals.
Fall River, Mass.	Dec. 8, 1874	25	2,500	488,000
Providence, R. I.	Sept. 27, 1877	25	9,500	....
Providence, R. I.	Feb. 15, 1888	22	4,400	2,000,000
Lynn, Mass.*	Nov. 26, 1889	20	6,700	3,000,000
Boston, Mass.†	Nov. 28, 1889	86	20,000	14,000,000
Milwaukee, Wis.	Oct. 28, 1892	66	21,200	9,500,000
San Francisco, Cal.	Nov. 1898	....	....	6,000,000
Jacksonville, Fla.	May 3, 1901	....	4,380	....
Atlantic City, N. J.	April 3, 1902	16	....	2,900,000
Boston, Mass.	Nov. 12, 1904	....	....	2,150,000
Baltimore, Md.	Feb. 7–8, 1904	100	20,000	70,000,000
Indianapolis, Ind.	Feb. 20, 1905	....	8,000	....

* There were also two broken 4-in. pipes discharging water.
† In this fire enough water was thrown over the burned district, which included 3½ acres, to have covered it with 12½ ft. of water.

406. Hydrant Pressure. — In the early water works of this country, the water pressure at the hydrants was usually about 40 to 50 lbs. per sq. in. With the use of steam fire engines, this pressure was sufficient for extinguishing fires in buildings that rarely had more than four stories. Modern water systems usually provide pressures of 60 to 80 lbs. at the hydrants. With 60-lb. pressure, the fire hose can be attached directly to the hydrants in the residential districts and in small cities, where no buildings have more than four stories, providing that the hydrants are not spaced too far apart. If a pressure of 80 lbs. can be maintained at the hydrants, fires in the commercial and manufacturing centers of most American cities can be extinguished by attaching the fire hose to the hydrants, if they can be reached with hose lines not exceeding 300 ft. in length. For longer lines, fire engines would be required, in order to obtain sufficient pressure at the base of the fire nozzle. With a water pressure of 100 lbs. and proper spacing of the hydrants, according to the fire risks in different parts of a city or town, fire engines can be dispensed with. With a pressure of 70 lbs. per sq. in. in the water mains, an automatic sprinkler supply (see Appendix) may be maintained in buildings up to eight stories in height.

Of late years, a number of large cities have constructed ***high pressure*** water systems (p. 410) having independent pipes for extinguishing fires in districts in which modern high buildings have been built. In some of the systems, a pressure of 300 lbs. per sq. in. can be maintained at the hydrants during fires.

The efficiency of a water works system for protection against fire depends largely upon the quantity of water a given group of hydrants can deliver during a fire. The manner in which this can be tested by means of Pitot gauges is explained on p. 401.

407. Steam Fire Engines. — When a steam fire engine — briefly called a *steamer* — is attached to a hydrant, the available pressure for extinguishing a fire can be much increased.

An interesting series of tests showing the service the different types and sizes of the steam fire engine of the Boston Fire Department could render were made by Dexter Brackett, M. Amer. Soc. C. E., in 1893.* The estimated capacity of these engines for 300 revolutions varied from 498 to 957 gals. per min. The water supply was obtained from a post hydrant, located at the end of an 8-in. pipe, 240 ft. long, which was connected with a 12-in. main. The hydrant had an inner diameter of 6¾ ins. and had three outlets, *viz.:* one of 2½-in. diameter and two of 4½-in. diameter, each outlet being controlled by an independent valve. When the engines were not drawing the water, the pressure at the hydrant was 42 lbs. Each of the steamers tested was connected with one of the 4½ outlets by an ordinary 4-in. suction hose. During most of the tests, the length of the hose was 500 ft., except when using a large Siamese nozzle, in which case the length of the line was only 200 ft. For a single line of hose, 1⅛-in. and 1¼-in. nozzles were used. For two lines siamesed into one nozzle, the diameter of the nozzles tested varied from 1⅜ to 2 ins.

These tests showed that the smallest fire engine was capable of delivering two good fire streams, each about 275 gals. per min., through 500 ft. of hose, and that the largest engine could deliver from 900 to 975 gals. per min. equal to three good 1¼-in. streams. In all of the tests the steamers obtained their water supply under a pressure of 30 to 40 lbs. If they had had to draw water entirely by suction, the fire streams would not have been so great.

The test also showed that when the length of the hose exceeded 500 ft., powerful fire streams could be obtained by using two lines of hose with siamese connections. The smallest engine with a single line of 2½-in. hose, 800 ft. long, and with a water pressure of 150 lbs. at the engine, had only a nozzle pressure of 30 lbs., while the same engine, with the same engine pressure, but using two lines of 2½-in. hose, siamesed into a

* Jour. New England Water Works Assoc., IX, p. 151.

50-ft. line with a 1$\frac{3}{8}$-in. nozzle, had a pressure of 55 lbs. at the nozzle. In the first case the stream delivered only 250 gals. per minute, and was not effective at about the third story. In the second case the delivery was 425 gals. per min., and the stream was effective 80 ft. above the ground. The tests showed that the efficiency of the fire engines depended principally upon the ability of the boilers to keep up the required steam pressure.

408. Flow Tests of Hydrants. — In testing the efficiency of a water works system in preventing fires, the quantity of water that can be obtained from a group of hydrants, in a given time, for extinguishing fires must be measured. The hydrants need only be kept open for a short time — say 3 or 5 minutes. This will be sufficiently long to determine the drop of pressure in the mains, caused by the opening of the hydrants. The length of time during which the flow can be maintained

Fig. 229. — Hydrant Pitot Tube.

can be determined by the available storage of water, the size of the pumps, and the size of the principal main from which the water must be obtained. In making these tests, usually 3 to 6 hydrants, and occasionally even more, are opened simultaneously, and a record of the variation of the pressure in the main is kept at some hydrant near the center of the group, by means of a pressure gauge. If the supply of water is ample, all of the outlets of the hydrants should be opened. If this causes too great a drop in the pressure of the main, some of the outlets must be closed, the object being to determine the quantity of water that can be maintained for fire purposes.

The discharge from each hydrant outlet can be measured accurately by means of suitable water meters, or by attaching to the outlet a short piece of fire hose, provided with a large smooth nozzle, the flow through which is measured by means of a Pitot tube and pressure gauge, in the manner done by John R. Freeman in his "Experiments Relating to the Hydraulics of Fire Streams." * A simpler and shorter method, which gives results that are probably correct within 5 per cent, has been developed by the engineers of the Committee of Fire Prevention of the National Board of Fire Underwriters, and has been used in testing hy-

* Trans. Am. Soc. C. E., 1889, XXI, p. 303.

drant streams in over 250 cities and towns. It consists in measuring, by means of a Pitot tube and pressure gauge, the average velocity of a fire stream issuing from an open hydrant outlet. This tube consists of two parts which are screwed together (Fig. 229). The part which is held in the stream is shaped like a blade, and is 4 or 4½ ins. long. One of its ends is closed, and a small inlet-tube is fastened near this end to the

Fig. 229*a*. — Gauging Flow of Hydrant with Pitot Tube.

blade and at right angles thereto. The second part of the Pitot tube is a piece of ¼-in. brass pipe, 8 or 10 ins. long, which forms the handle of the instrument. A pressure gauge is attached to the handle at its open end. By placing a union in this connection, the joint may be kept tight in whatever position the gauge may be held.

In using this Pitot tube, the inlet-tube is moved to different points in the cross-section of the fire stream with a view of determining the average velocity of the stream (Fig. 229*a*). For small outlets, 2½ ins. or less in diameter, it is only necessary, in ordinary cases, to measure the maximum velocity of the stream at the center of the outlet. By multiplying this velocity by a coefficient found by experiments, the average velocity is obtained. For larger outlets the average velocity of the fire stream can be found with sufficient accuracy by taking the average velocity recorded at the center of the outlet and at each end of its vertical and horizontal diameter. In measuring these velocities, the inlet of the Pitot tube should not be placed nearer than $\frac{1}{4}$ in. from the sides of the outlet, as the velocity diminishes very rapidly at the sides, owing to friction. A little practice with these instruments soon makes a person expert in using them.

When the pressure at a hydrant is low, the stream flowing from it may not fill the whole area of the outlet. In such cases the *area of no flow* is generally a segment at the bottom of the outlet. Any projection into the stream, such as the stem of an independent valve, or the roughness of the hydrant nipple, will produce small *holes* in the stream. The *areas of no flow*, which can usually be measured with sufficient accuracy by means of an ordinary scale, must be deducted from the total area of the outlet, found by careful calibration, before multiplying by the average velocity of the stream, in order to obtain the discharge.

The most convenient pressure gauge for use with the Pitot tube described above is a 3-in. gauge, graduated in half pounds, from 0 to 50 lbs. This gauge can be easily read to the nearest quarter of a pound. Before using the pressure gauge, it should be tested by means of a *weight tester* made for this purpose, or by comparison with an accurate test gauge, or with a water column.* In making these tests the gauge should be, as nearly as possible, in the condition in which it is to be used, as regards temperature, exposure to the sun, etc. A gauge may be very accurate in a cool drafting room, but may give very different results when exposed to the sun. For low pressures of a pound or less, the gauges mentioned above are not always sensitive enough. In such cases the pressures can be easily and accurately found by replacing the gauge by a small rubber tube, full of water and open at the top, which must be held at the elevation at which the water begins to overflow the tube. By measuring the vertical height of the top of this tube above the center of the hydrant outlet, the pressure at the hydrant can be easily found.

In using the method of measuring fire streams at hydrant outlets, described above, *i.e.*, by multiplying the area of flow by the average

* Emil Kuichling, M. Am. Soc. C. E., while Chief Engineer of the Rochester Water Works, made careful tests of the best pressure gauges he could obtain in the market, by comparing them with a water column, the height of which was varied. He found many of the gauges to be inaccurate, especially for low pressures.

velocity found by the Pitot tube, the results obtained are somewhat greater than the actual discharges, and the quantities obtained must be multiplied by a coefficient found by experiments. By accurate tests made on three different makes of hydrants, with outlets of various sizes and with velocities of flow caused by pressures varying from $\frac{1}{2}$ to 28 lbs., the engineers of the National Board of Fire Underwriters found the actual discharges to be 0.85 to 0.96 of the theoretical quantities, the average being about 0.90. In these experiments, the actual discharges were measured by means of a Venturi meter, a Worthington current meter, and by measuring the flow in the supply pipe with a pitometer. The coefficient of 0.90 can be safely used in the above described method of finding the discharge from a hydrant, after proper deductions have been made for areas of *no flow*.*

409. Freeman's Experiments with Fire Streams. — In 1888 John R. Freeman made a series of experiments for the Associated Factory Mutual Companies of New England to determine:

1. The discharges from various kinds of fire nozzles under different pressures.
2. The losses of pressure caused by friction in different kinds of fire hose, and by the curves and sinuosities of a hose line.
3. The loss of pressure resulting from reducing the water way at the coupling of a hose.
4. The effective reach as regards height and distance of fire streams from various kinds of nozzles, under different pressures.
5. The distribution of velocity in jets from fire nozzles.

The results of these experiments were given by Mr. Freeman in detail in a paper on "Experiments relating to the Hydraulics of Fire Streams" published in Trans. Am. Soc. C. E., 1889, XXI, p. 303,† with numerous practical tables.

Mr. Freeman's experiments were made with the utmost care. The pressures in the hose line were measured accurately, by means of two light, portable, open, mercury columns, each about 20 ft. high, and capable of registering pressures up to 120 lbs. per sq. in. One of these columns was placed about 5 ft. from the hydrant from which the water for the experiments was obtained, and the other was attached to the hose line at the base of the fire nozzle. In order to obtain, as nearly as possible, the average pressure in the hose at the points where the mercury columns

* The author is indebted to George W. Booth, Chief Engineer of the Committee on Water Supply of the National Board of Underwriters, for the facts stated above, and for other valuable information about fire protection.

† A shorter account of these experiments is given in the Jour. New England Water Works Assoc., 1889, IV, p. 95.

were placed, these columns were attached to hollow couplings placed in the hose line, each having four small holes at the extremities of two diameters at right angles to each other. A timber tank lined with sheet zinc, having a capacity of 1450 gals., was used for measuring the discharges of the different fire streams tested in the experiments. During the experiments, the pressure in the water main was about 75 lbs. at the hydrant, and it was forced up at times to 130 lbs., by means of an engine.

410. *Fire Nozzles.* — About forty different kinds of nozzles, varying in diameter from $\frac{3}{4}$ to $1\frac{3}{4}$ ins., were tested, in order to find the one that gave the best jet. Smooth nozzles (Fig. 230) were found to give jets a little superior to those of ring nozzles (Fig. 230*a*). A smooth coni-

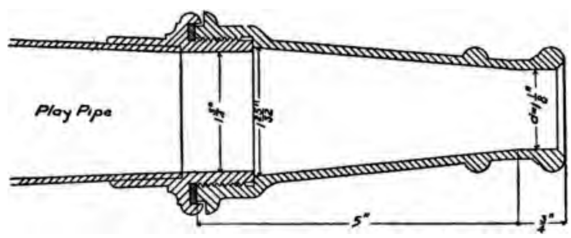


FIG. 230. — Smooth Nozzle.

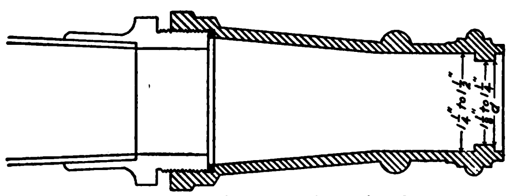


FIG. 230*a*. — Square Ring Nozzle.

cal nozzle gave the best results. Such a nozzle was designed by Mr. Freeman after his experiments had been completed. In jets from the best nozzles, the velocity of the water was found to be uniform throughout the cross-section of the jet, except in the immediate vicinity of the walls of the orifice. The actual velocity of discharge in the central portion of jets from sharp-edged and conical orifices was equal to the theoretical velocity, but the mean velocity of the whole jet was always slightly less than the theoretical velocity, owing to the retardation caused by the walls of the orifice. The average coefficients of discharge for the two principal kinds of nozzles based upon Freeman's experiments, are: 0.974 for smooth nozzles, and 0.740 for ring nozzles.

411. *Fire Hose.* — The tests included hose made of leather, solid rubber, unlined linen, and various kinds of rubber-lined hose. Most of it was nominally $2\frac{1}{2}$ ins. in diameter, the actual diameter varying from

$2\frac{7}{16}$ to $2\frac{3}{4}$ ins. Some tests were made with 2-in. and $2\frac{1}{4}$-in. hose. The pressure in the hose varied from 15 to 125 lbs., and the deliveries from 50 to 325 gals. per min.

The character of the inner surface of a hose, as regards smoothness, was found to have a very material effect on the discharge. Curves of any radius, possible without cramping the hose, had but a trifling effect in retarding the flow. The usual sinuosity of a hose line was found to increase the friction in the hose only about 5 per cent. The loss of pressure caused by suddenly reducing and then enlarging the area of the water way, by using couplings having diameters slightly smaller than that of the hose, was found to agree fairly well with the theory that this loss equals the head corresponding to the difference in velocity due to passing from a contracted channel to a larger one.

412. Dexter Brackett's Experiments on Loss of Pressure in Fire Hose. — In 1893 Dexter Brackett, M. Am. Soc. C. E., made some experiments * in Boston to find the loss of pressure caused by friction in fire hose and, also, to ascertain the effect of using $2\frac{1}{2}$-in. couplings on a line of 3-in. hose. For the latter purpose, $2\frac{1}{2}$-in. bushings were placed at each coupling of the 3-in. hose, to obtain the same effect that would be caused by using $2\frac{1}{2}$-in. couplings. In all of these tests the lines of hose were 500 ft. long. With the same water pressure at the steamer, and each line having a $1\frac{1}{4}$-in. nozzle, the pressure at the nozzle of a line of 3-in. hose was found to be 12 to 15 lbs. greater than that obtained with a $2\frac{1}{2}$-in. hose. When a $1\frac{1}{4}$-in. nozzle was used for the 3-in. hose, and a $1\frac{1}{8}$-in. nozzle for the $2\frac{1}{2}$-in. hose, both nozzles had the same effective pressure, but the delivery of the former exceeded that of the latter by 20 to 25 per cent.

The use of a $2\frac{1}{2}$-in., instead of a 3-in., coupling in a line of 3-in. hose caused a loss of about $\frac{1}{2}$ lb. at each coupling when discharging 475 gals. per min. The friction loss on the hose itself was found to be about as follows:

Description	250 gals. per min.	300 gals. per min.
	Lbs.	Lbs.
$2\frac{1}{2}$-in. hose	13	18.0
3-in. hose, $2\frac{1}{2}$-in. couplings	8	10.7
3-in. hose, 3-in. couplings	8	10.5

The following test shows the advantage of using 3-in. hose for long lines:

With a steamer using a line of $2\frac{1}{2}$-in. hose, 600 ft. long, and a $1\frac{1}{4}$-in.

* Jour. New England Water Works Assoc., 1894, IX, p. 151.

nozzle, 300 gals. per min. was discharged, the water pressure at the steamer being 150 lbs. When a line of 3-in. hose was substituted for the $2\frac{1}{2}$-in. hose, only 105 lbs. pressure was required at the steamer, in order to make the same delivery. The 3-in. hose is too heavy to be handled without difficulty on ladders and in buildings, but is of great use on long lines of hose, laid in the street, if one or two lengths of $2\frac{1}{2}$-in. hose are attached for the use of the hoseman. When both of these sizes may be used, it is an advantage to use only $2\frac{1}{2}$-in. couplings, to facilitate making connections, as the loss of pressure they cause, placed in a line of 3-in. hose, is insignificant.

413. *Loss of Pressure in a Hydrant.* — During his tests of fire streams, Mr. Brackett ascertained the loss of pressure caused by the hydrant from which he obtained the water for these trials. As stated on p. 400, it was a post hydrant, located at the end of an 8-in. pipe. It had an inner diameter of $6\frac{3}{4}$ ins., a 6-in. rubber valve and three outlets, each controlled by an independent valve. Two of these outlets had a diameter of $4\frac{1}{2}$ ins., and the third had a diameter of $2\frac{1}{2}$ ins. The loss of pressure caused by the hydrant was determined by recording the pressure shown by a gauge attached to the 8-in. pipe, at the base of the hydrant, and that indicated by a piezometer, attached to the suction hose of the steamer. When the entire discharge of the hydrant was taken from one of the $4\frac{1}{2}$-in. outlets, the friction loss in the hydrant was 4 lbs. for a delivery of 500 gals. per min., and 16 lbs. when the delivery was doubled. A large proportion of this loss was found to be caused by the outlet valves.

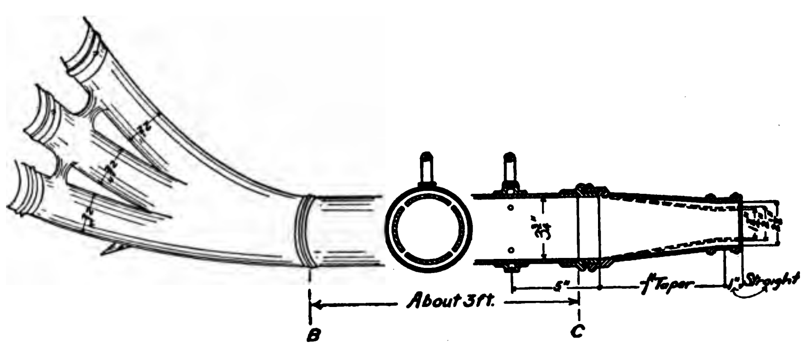


FIG. 231. — Freeman's Siamese Nozzle with Piezometer Connection.

414. The Nozzle as a Water Meter. — The experiments made by Mr. Freeman showed that small quantities of water could be measured as accurately with a fire nozzle as with a weir. The former can be used in this manner for measuring the delivery of a pump. Mr. Freeman designed for this purpose the Siamese nozzle, shown in Fig. 231. It has a $3\frac{3}{4}$-in. barrel and three $2\frac{1}{2}$-in. connections. This nozzle will measure, within about $\frac{1}{2}$ per cent, a flow of 1400 gals. per min., equal to about 2 million gallons per day. It weighs about 75 lbs. and costs about $100. In using this nozzle it should be attached to new rubber-lined hose. If possible, the pressure should be measured for accurate experiments by a mercury column. For ordinary practical purposes, a good Bourdon duplex spring gauge, tested before being used, will answer. A mistake of 2 per cent, caused in reading the pressure by such a gauge, would only make an error of about 1 per cent in the quantity of the discharge.

415. Location of Hydrants. — The spacing of the hydrants for proper fire protection depends upon the nature of the locality, and upon the pressure that can be maintained at the hydrants during fires. As a hydrant may be out of service, on account of being frozen, or out of order, at least two hydrants should be available for putting out a fire that may occur in a small town or in the suburbs of a large city. Practical experience shows that single hose lines should not exceed a length of about 500 ft. for good service. For greater lengths, it is advisable to use two lines of hose and to siamese them into a single line near the fire nozzle. In a residential or suburban district, the distance from hydrant to hydrant should be about 300 to 500 ft. In the closely built-up parts of a city or town, this distance should not exceed 150 to 250 ft. The linear spacing of hydrants does not, however, give a correct idea of the fire protection they afford, as this evidently depends also upon the depth of the lots that have to be protected by fire hydrants placed along their front. With deep lots, more hydrants will have to be placed in the streets bounding them than would be required for lots of less depth. Basing the placing of the hydrants upon the area to be protected, instead of upon the length of mains, a safe fire protection is afforded by having one hydrant per acre in closely built-up districts, and one hydrant per three acres in residential and suburban sections.

416. Freeman's Tables for Fire Streams. — Based upon his experiments, Mr. Freeman prepared a series of practical tables for fire streams for nozzles varying from $\frac{3}{4}$ to $1\frac{3}{8}$ ins. in diameter, for pressures from 5 to 100 lbs. per sq. in. at the base of the play-pipe. These tables give the discharges of different sizes of nozzles, under different pressures, and the pressure lost by friction in $2\frac{1}{2}$-in. fire hose, from 50 to 1000 ft. in length. In computing these tables the coefficient of discharge for a smooth nozzle was assumed at 0.974 and the following values were adopted in figuring the losses caused by friction, including an allowance for ordinary curves and for ordinary slight excess of actual over nominal diameter.

Loss of Pressure in 100 ft. of $2\frac{1}{2}$-in. Fire Hose

	Lbs.
Linen hose	30
Inferior rubber-lined hose	26
Best rubber-lined hose	13

417. Fire Stream Tables of the National Board of Fire Underwriters. — We give in the Appendix practical data connected with fire streams, based upon the experiments of John R. Freeman (Trans. Am. Soc. C. E., Vols. XXI and XXIV); and upon tests of rubber-lined fire hose made in October, 1909, by the engineers of the National Board of Fire Underwriters, with the assistance of the New York Fire Department, in coöperation with the engineers of the Department of Water Supply, Gas and Electricity of the City of New York. These tables are reproduced by the courtesy of the National Board of Fire Underwriters from a booklet it published in 1912 on "Fire Engine Tests and Fire Stream Tables." They give the discharge of smooth nozzles, 1 to $2\frac{1}{2}$ ins. in diameter, for pressures varying from 20 to 100 lbs. per sq. in. For fresh water the discharge for any size nozzle can be determined by the following formula:

$$\text{Gallons per minute} = 29.83\, cd^2 \sqrt{p}, \tag{100}$$

where d = diameter of nozzles in inches, measured to $\frac{1}{1000}$ of an inch;
p = pressure in pounds per square inch, recorded on Pitot gauge;
c = a constant, varying from 0.990 for 1 in. nozzles to 0.997 for 6-in. nozzles.

For ordinary use the formula can be reduced to

$$\text{Gallons per minute} = 29.7\, d^2\sqrt{p}. \tag{101}$$

CHAPTER XX

HIGH-PRESSURE WATER SYSTEMS

418. Necessity of High-pressure Water in Cities. — The value of buildings and property in large cities in the United States has grown so rapidly of late that the question of proper protection against fire has become of paramount importance. Great conflagrations within the past 20 years have shown the inefficiency of the present systems of protection. What has made the extinguishing of fires, in many cases, very difficult, is the great height of modern buildings, which necessitates an increase in pressure in the fire streams.

419. Separate Water Supply Systems for Fire Protection. — In some of the cities situated on the seacoast, on lakes or rivers, powerful fire-boats have been maintained as auxiliaries to the steam fire engines. These boats are of great help in extinguishing fires that occur near the water front. To make them available for fires further inland, some cities began, about 25 years ago, to lay special lines of pipe from the water fronts to important business sections, and to make connection to enable the fire-boats to pump water directly into these pipe lines. This was a step towards proper protection against fire, but in winter, ice occasionally prevented the fire-boats from being able to connect to the special line of pipe. For this reason permanent pumping stations for fire protection were gradually built, to replace the service from fire-boats, and the lines of special pipe for protection against fire were extended until they formed independent systems of pipe, used only for extinguishing fires. Some cities * have gravity supplies giving pressures of 116 to 180 lbs. per sq. in., which is sufficient in many places for proper protection against fire, but in the majority of cases the pressure desired, usually 150 to 300 lbs. per sq. in. at the hydrants, must be obtained by pumping with powerful engines or by building high-service reservoirs. The latter method was adopted for the "high-pressure" pipe system of San Francisco.

Cleveland, Detroit, Milwaukee, Boston, and Buffalo were among the first cities in the United States to lay and maintain independent fire service mains.

The special provisions made for protecting buildings, such as automatic sprinkler systems, etc., are described in the Appendix.

* Providence, R. I.; Lawrence, Mass.; Worcester, Mass.; Newark, N. J., etc.

420. High-pressure Water System of New York. — The city of New York made a beginning of a *high-pressure system* in 1906 at Coney Island, a pleasure resort on the Atlantic Ocean, where all buildings were built of highly inflammable materials. This was followed in 1908 by a system for protecting the most important business, manufacturing, and water front sections of the borough of Brooklyn, and in 1906 to 1909 a *high-pressure system*, the greatest of its kind, was constructed to protect the southerly part of the borough of Manhattan, where more concentrated value of property exists in a relatively small area than, probably, in any other city. The high-pressure system was gradually extended. As it is one of the latest and most important of its kind, we shall describe it somewhat in detail.

421. *Pumping Stations.* — The territory covered by the *high-pressure system* contains about 1400 acres, and includes the most important wholesale manufacturing and warehouse sections, the largest docks, and, also, some of the most thickly populated tenement house districts. The water required for this system is taken, except in cases of emergency, from the regular city supply and is pumped at two stations, one located on the North River and the other on the East River, into the high-pressure pipe system. Each of these stations is supplied by a 48-in. and a 36-in. main. Should this supply be unavailable for any reason, salt water can be pumped direct from the rivers that surround Manhattan Island.

Each station contains six-stage centrifugal pumps, directly connected to 3-phase, 25-cycle electric motors, driven by a current of 6600 volts at a speed of about 750 r.p.m. The two stations together can deliver a maximum of 36,000 gals. per min., under a pressure of 300 lbs. at the hydrants. All of the pumps can be put into instant operation by simply throwing a switch, which turns on the electric current, and the maximum pressure of 300 lbs. per sq. in. at the hydrants is obtained in about a minute.

422. *Pipe System.* — The water is distributed through the system by 24-in. mains, two coming from each pumping station. These trunk pipes practically encircle the entire area that is to be protected, and are supported at short intervals by 20-in. mains (Fig. 232). The latter are cross-connected by 16-in. mains which supply 12-in. pipes, the smallest size used in the system. By the system described, the loss due to friction in the pipes between the pumping stations and any particular hydrant is reduced to a minimum. Under the least favorable conditions, the loss due to friction, between the pumping stations and the remotest hydrant, does not exceed 25 lbs. per sq. in., under maximum discharge.

The pipes and specials used in this high-pressure system are all of cast iron with the exception of the larger 3-way and 4-way branches, which are made of cast steel.

The thickness of metal in the pipes varies from $\frac{7}{8}$ in. in the 8-in. hydrant branches to $1\frac{7}{8}$ in. in the 24-in. mains. Hub-and-spigot pipe-joints, filled with lead and caulked in the usual manner, are used. Two grooves

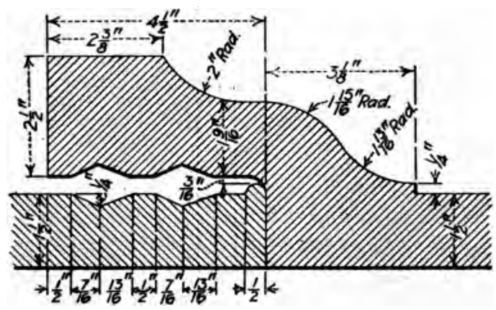


FIG. 232. — Pipe-joint for 20-in. Main (New York High-pressure Fire System).

are provided in each hub, to give the joints increased strength to resist being pulled apart. At first these grooves were made semicircular, with a diameter of $\frac{5}{8}$ in., but later they were made triangular, as this shape gives a greater area of lead to resist shearing, without adding to the quantity of lead required for each joint. All pipes, specials, and other fittings were tested at the foundry to a pressure of 650 lbs. per sq. in., and after being laid, they were subjected to a pressure test of 450 lbs. per sq. in. The experience with these pipes has been very satisfactory, in every respect, and no break has thus far occurred.

423. *Hydrants.* — The hydrants used in this system are of the post type, and are made according to a special design, adopted after exhausted tests. They are spaced so as to give one hydrant to every 220 ft. of main, averaging 0.94 to the acre. This arrangement makes it possible to concentrate twenty 2-in. fire streams on any city block. These streams, with a nozzle pressure of at least 75 lbs. per sq. in., discharge about 20,000 gals. per min. The branch-pipes from the mains to the hydrants are all 8 ins. in diameter. The barrel of the hydrant has an inner diameter of $10\frac{7}{16}$ ins. The hydrant has four 3-in. outlet nozzles, each provided with an independent slide valve.

Up to January 1, 1917, 128 miles of pipe and 2751 hydrants have been placed in the high-pressure fire system of the borough of Manhattan.

424. *Operation.* — In operating the system, a force of twelve men, divided into three shifts, is required at each pumping station. Each shift consists of an engineman, an oiler, a laborer, and a telephone operator. A normal pressure of 30 lbs. per sq. in. is maintained in the pipe system through a by-pass connection from the domestic supply mains, controlled by a check-valve and an electrically operated gate-valve. This pressure can be immediately raised to 125 lbs. per sq. in., upon receipt of a fire alarm, which is sounded simultaneously at the headquarters of the Fire Department, and at the two pumping stations. Upon receipt of special orders from the Chief of the Department, the pressure can be further increased. A separate telephone system, independent of the fire alarm circuits, has been established in the high-pressure district. It includes 371 telephones, placed in boxes on the walls of buildings, distributed over the area. This enables the official in charge of a fire to communicate easily with the pumping stations.

On January 7, 1909, the high-pressure system of Manhattan was given a severe test, which it stood with perfect success. Three large dangerous fires, and two smaller ones, occurred simultaneously within the high-pressure district. To master these fires, forty companies of firemen, numbering in all 12 battalion chiefs and more than 600 men, had to be called out. Seven of the centrifugal pumps at the two stations had to be put into service, and delivered water at the rate of 33,500 gals. per min., with a pressure of 225 lbs. per sq. in. at the stations, and of 205 lbs. at the hydrants. The total quantity of water used to extinguish these fires amounted to 14 million gallons, a quantity sufficient to have flooded the area of the buildings on fire to a height of 70 ft.

425. *Reduction of Fire Insurance Rates.* — Since the high-pressure system has been established on Manhattan, there has been a great reduction in fire insurance rates, the total saving in insurance being estimated by conservative insurance men to be now about $650,000 per annum. In connection with this system, the City has been able to effect a considerable saving in the Fire Department, by dispensing with the costly and unwieldy portable fire engines, substituting therefor simply a sufficient number of hose carts.

426. *Engineer.* — The high-pressure water systems of the boroughs of Brooklyn and Manhattan of the city of New York were designed and executed under the direction of I. A. De Varona, Chief Engineer of the Department of Water Supply, Gas and Electricity of the City of New York.

427. The First High-pressure Fire System of Philadelphia, Pa.,* was constructed in 1904 to 1911, to protect the congested district between the Delaware River and Broad Street, and between Race and Walnut Streets, containing about two-thirds of a square mile. The water required for the high-pressure system is pumped from the Dela-

* Proc. Engineers' Club of Philadelphia, 1903, XX, pp. 54 and 83.

ware River at a station, built at the foot of Race Street, where pumps having a capacity of 9100 gals. per minute are installed.

428. *The pipes,* which are 8 to 20 ins. in diameter, are made of cast iron and are provided with flanges. They are laid 6 ft. below the surface of the streets, to have ample protection against freezing. One-inch air-valves are placed at all summits. For about 36 miles, *Universal Pipe*, p. 83, was used. The pipes were required to stand a test pressure of 800 lbs. per sq. in. at the foundry. After being laid, the pipes were tested in lengths of 100 to 200 ft., under a hydraulic pressure of 400 lbs. per sq. in. No allowance was made for expansion and contraction. After the pipes had been laid, some lengths of pipe broke, near the flanges. This trouble always occurred near the summit of a grade and immediately under the flange of a pipe, and was thought to be due to the drag of the long length of pipe upon the grade. In order to avoid this trouble, expansion joints were placed in the different pipe-lines, about 400 to 500 ft. apart. These joints consist of sleeves, surrounding the pipe. The annular space between the pipe and sleeve was filled with 16 strands of $\frac{1}{8}$-in. lead wire, driven cold. Each strand was one inch longer than the circumference of the pipe and was plated by hand, put in the bell and driven home, against a packing of hemp, which was dipped in North Carolina tar. This process was repeated until a point was reached, one inch from the outer edge of the bell, the last inch being filled with molten lead. These joints, when tested under a pressure of 400 lbs. per sq. in., were found to be perfectly water-tight. Each of the joints was placed in a manhole, large enough to permit recaulking when necessary.

429. *The fire hydrants* are of the *flush type*. They are placed at the corners of each intersecting main street, and one or more near the middle of a square, at the corner of an intermediate street or streets. This provides from 12 to 16 fire hydrants for each square, and from 32 to 48 hose attachments, all of which can be concentrated upon one building in an emergency. Each hydrant has an internal diameter of $7\frac{1}{4}$ ins., and is provided with two 4-in. outlets, each having an independent valve. The hydrant is controlled by a gate placed upon its connection with a main pipe, and is drained automatically through a 1-in. galvanized pipe to a near-by sewer. The hydrants — each of which weighs 1000 lbs. — as, also, the branch-pipes and gate-valves, are made of semi-cast steel, a material consisting of melted cast iron to which steel scrap has been added.

430. *Fire-boats.* — As sufficient funds for completing the whole high-pressure fire system, which was estimated to cost $600,000 to $700,000, were not available when the works were originally built, reliance was placed, as a temporary expedient, upon the fire-boats until the proposed pumping station should be completed. The boats available for this purpose were: A fire-boat having four pumps, each with a capacity of 2240 gals. per min., and five police boats, each having a capacity of 750 to 1300 gals. per min. In case of fires, the boats pump into a Siamese connection at the water end of a fire main. Each of these connections is provided with eight hose connections.

A second high-pressure system, covering an area of about 3.5 square miles, was built in 1910 to 1912 to protect the most important plants in the Kensington manufacturing district. This territory, which lies north of the first high-pressure district, is supplied by the Fairhill pumping station, which draws water from the Fairhill Reservoir. The total capacity of this station is 12,350 gals. per minute.

431. *Engineer.* — The first high-pressure water system was designed and constructed under the direction of F. L. Hand, Chief Engineer of the Bureau of Water.

432. High-service Fire System of Baltimore, Md.* — In 1904 a great conflagration occurred in Baltimore, and demonstrated that while there was enough water in the reservoirs, the existing fire fighting equipment of the city was inadequate. The fire raged for thirty hours and covered an area of 150 acres, causing a loss of about $100,000,000. In order to prevent a repetition of such a fire, a high-pressure water system for protection against fire was constructed in Baltimore in 1904 to 1912.

The surface of Baltimore varies in elevation from 6 to 460 ft. above mean low water. In order to avoid excessive pressures in low lying sections, the water supply of the city is divided into five separate services. The two lowest services receive water by gravity, and the three high services are supplied by pumping, and from high storage reservoirs. Suitable by-passes connect the services, so that, in an emergency, any service can be supplied from the next higher one. The high-pressure fire system was constructed to protect a congested district in the lower and middle service zones, covering about 300 acres, and rising 6 to 100 ft. above mean tide.

The high-pressure system of Baltimore was built after those of New York and Philadelphia had been begun. It differs from these two systems by the use of steel water mains, flush hydrants, portable hydrant heads, and steam engines at the pumping stations.

433. *The distribution system* forms a grid iron of 16-in. mains, three blocks (about 1200 ft.) apart in both directions, with 10-in. laterals in the intermediate streets. All hydrant branches are 8 ins. in diameter.

* Trans. Am. Soc. Mechanical Engineers, 1913, XXXV, p. 183; *Insurance Engineering*, August, 1912, p. 72.

There are no *dead ends* on mains or laterals in the entire system. Two 24-in. mains from the pumping system discharge into the intersections of the 16-in. pipe. They are looped around the rear of the pump foundations in the pumping station, in order to avoid *dead ends* and, by equalizing the stresses, to make heavy anchorages unnecessary. The pumps in the station discharge through 14-in. pipes into the 24-in. mains, each of which is provided with a large rolled steel air-chamber, 30 ins. in diameter and 20 ft. high. By means of two 24-in. gate-valves and an 18-in. cross-connection in the pumping station, any section of the 24-in. mains can be cut off, without putting more than one pump out of commission. A 10-in. branch is provided at the harbor front for connection with fire-boats.

434. *Pipes.* — All of the water mains are made of lap-welded, soft open-hearth steel. This material was selected, in preference to cast iron, for these mains, on account of its greater strength, which renders it less liable to breakage, and makes it possible to space the pipe-joints farther apart.

The standard length of the pipes is 20 ft., which reduces the number of joints 40 per cent from what is required with cast iron pipes having the usual length of 12 ft. A thickness of metal of $\frac{7}{16}$ in. was specified for the 8-, 10-, and 16-in. mains, but a thickness of $\frac{1}{2}$ in. was required for the 24-in. pipes.

Great pains were taken to make the coating of the pipes as perfect as possible, so as to avoid corrosion. The specifications for the asphalt required that a cubic centimeter of the asphalt should show no action when exposed for one year in any or all of the following solutions: 25 per cent hydrochloric acid; 25 per cent sulphuric acid; 25 per cent potassium cyanide; 25 per cent caustic soda; saturated solution of ammonia.

The pipes, after being thoroughly cleaned, were heated to a temperature of 300° F., and dipped vertically into a bath of asphalt, which was maintained at a temperature of 350° to 400° F. Each pipe was held in this bath for a sufficient length of time, and was then drawn out slowly at the rate of about 5 to 10 ft. per min. By this treatment, an evenly distributed asphalt coating $\frac{1}{32}$ in. thick was obtained. Injuries to this coating, caused by the handling of the pipes, were repaired by applying, several times, the asphalt, dissolved in a suitable solvent, until a satisfactory thickness of coating was obtained. All nuts and bolts used in the joints of the pipes were dipped in the same solution of asphalt, before being inserted in the flanges of the pipes.

The effects of electrolysis were avoided by the coating described above, and by making the electrical conductivity of the joints approximately equal to that of the body of the pipe. At several points, the pipe was bonded to return feeders. The electrical surveys, which have been made regularly since this system was installed, show that the pipes are negative to the rails throughout.

435. *The Pipe-joints.* — In designing the joints for the pipe-system, rubber gaskets were avoided, because they would increase the electrical resistance at each joint. Gaskets of copper, or of other metals differing from that used in the pipes, were not used on account of the liability of galvanic action taking place, when the different metals should be placed in damp ground, and causing corrosion. The joint decided upon is shown in Fig. 88, p. 150. It is a universal joint which is formed as follows:

A loose flange of medium open hearth steel is slipped over the pipe at each end. The pipe is flanged out at its ends into a bell formed zone of a sphere. The pipes are separated by rings of soft cast steel, which are turned so as to fit accurately the bell-ends of the pipes. When two lengths of pipes are to be connected, the separating ring is heated to a temperature which is just sufficient to melt the asphalt coating of the pipes, so as to bring metal to metal. The ring is then held in place by means of a rod screwed into a small tap-hole, provided at the peak of the ring, and the pipes are brought together so as to bear firmly against the ring. By means of flange-bolts, the pipes are screwed tight together. The joint which is thus obtained is absolutely water-tight, until pressures are reached which exceed the elastic limits of the pipes or the bolts.

On account of the flexibility of this joint, ordinary obstructions in the pipe-trench can be avoided without the use of specials. The joint is designed for a deflection of 10 degrees — about $3\frac{1}{2}$ ft. in 20 ft. This is easily obtained with the 10-in. pipes, but for the larger size pipes is made a little less. For greater deflections bends are used which have generally a standard radius of five diameters of the pipe. The bends for the 24-in. mains were made at the factory but those for the pipes of less diameter were made in the field.

436. *Pipe Fittings.* — The only specials used in the fire system were the tees and crosses at the intersections of the gridiron, and valve connection pieces which were required to give a straight face-flange at the valves. All of the fittings were made of low carbon, open-hearth steel.

The hydrant branches were made by welding necks to the mains and laterals in the following manner: A hole, smaller than the size of the neck, was cut in the water main, and radial cuts were made, forming four narrow lugs, which were left projecting into the hole. The wider, alternate lugs were bent back to make an opening large enough to receive the neck-piece (Fig. 233). The smaller short lugs formed a support for the neck and held it rigidly in place, during the welding operation. The whole joint was then flowed with metal by means of an oxy-acetylene blow pipe. The joint formed in this manner is as strong as the original pipe.

437. *The gate-valves* are made of semi- and open-hearth steel, according to a substantial double-disc type. They are bronze mounted and

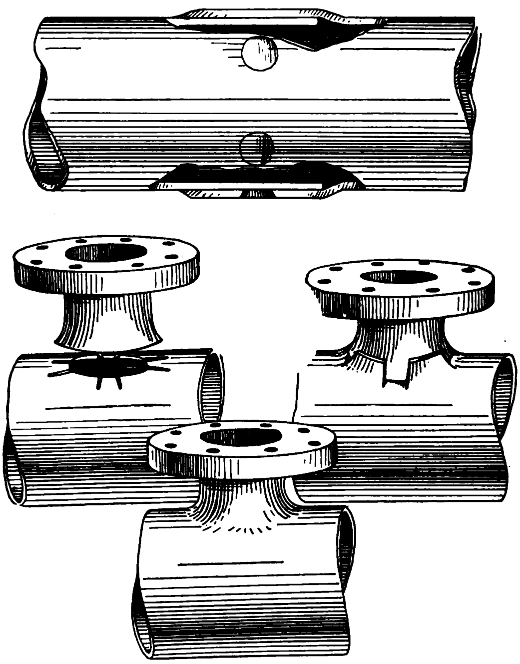


FIG. 233. — Branch Connection, Baltimore High-pressure Water System.

are painted inside and outside with asphalt paint. They were tested to a pressure of 600 lbs. per sq. in. when closed, and to 800 lbs. when open. The 16- and 24-in. valves are geared, and the latter are provided with 4-in. by-passes. Each street gate-valve is supplied with a stem-nut, and a forged steel key is placed in each valve-box. Owing to an interlocking arrangement, the key can only be removed when the valve is wide open. The valves are placed in concrete valve-boxes or in manholes, 42 ins. in diameter, and are generally located on property lines, ordinarily four at each street intersection. The average length of pipe between valves is 300 ft., and the maximum length is 480 ft., with the exception of the 24-in. line.

438. *Relief- and air-valves* are installed at several places in the distributing system, and their discharges are piped to the nearest sewer.

The former are 6 ins. in diameter, and have nickel seats. They are spring loaded, and are set to open at 305 lbs. per sq. in. Air-valves are placed at all high points of the pipe-system to discharge accumulations of air.

439. *Flush hydrants* are used in the entire fire-system, as they can be placed, almost without restriction, in the driveway or any part of the footway. Each hydrant consists simply of a piece of 10-in. pipe, made of cast steel, and resting on a concrete base in a concrete chamber. It is connected with the water main by an 8-in. branch, in the manner described above, and has a gate-valve at its base. Before opening this

FIG. 234. — Hydrant Head, Baltimore, Md.

valve, the water is first by-passed around it by means of a pilot-valve, and fills the hydrant, thus equalizing the pressure on both sides of the gate-valve. The operation of closing this valve opens a small valve in a drain-pipe, which permits the water in the hydrant to drain out to the level of the main gate-valve, which is always placed well below the frost-line. With this arrangement frozen hydrants are an impossibility. The hydrant is open on top, and is closed by means of a portable hydrant head, when it is to be used. When not in use, a small cast iron cover is laid over the top of the hydrant barrel, to protect it from dirt and injury, and over this is placed a larger cast iron cover, flush with the pavement or flagging. As practically all of the hydrants are placed in the sidewalks, this cast iron cover is made so light that it can easily be broken by a blow of the operating key, in case it should be frozen in its seat. The hydrants are placed about 170 ft. apart, and each serves an area of about 42,700 sq. ft. The location of each hydrant is indicated by markers on the trolley wire poles.

440. *The portable hydrant-head* * (Fig. 234) is one of the characteristic features of the Baltimore fire-system. It weighs 110 lbs. and can easily be attached to the hydrant barrel by one man. This is done by means of the bayonet joint shown in Fig. 235. The head slips loosely

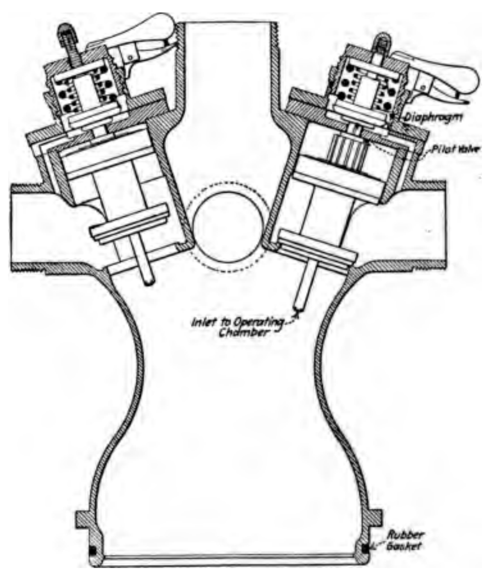


Fig. 235. — Section of Baltimore Hydrant Head.

into the barrel of the hydrant, and by a twist of $22\frac{1}{2}$ degrees, a series of interlocking lugs on the head and barrel engage each other. The lugs are made strong enough to resist in shear the full water pressure. A tight water joint is obtained by means of a square, soft rubber packing ring, which is placed in a square groove, somewhat larger than the rubber ring, on the outside of the lower portion of the head. At low pressure some water leaks past the rubber ring, but at higher pressures the ring is forced against the barrel, and makes a joint that is absolutely watertight, under pressures varying from 20 to 1000 lbs. per sq. in. When the water is shut off, the ring contracts and permits the hydrant head to be easily removed. The action of this water-joint is perfectly automatic.

* The portable hydrant-head used on the Baltimore Water Works is made by the Ross Valve Manufacturing Co., Troy, N. Y.

The portable hydrant-head is provided with four lateral, $2\frac{1}{2}$-in. hose outlets, and with one 3-in. opening in the top of the head for a monitor nozzle. The flow through each of these outlets is controlled and regulated by means of a balanced valve, operating in a chamber, built transverse to the water outlet, and actuated by the flow of a small portion of the water in the head. The chamber is closed by a plate diaphragm carrying a pilot-valve and a guide for the main valve. The diaphragm is covered by a spring chamber in which two spiral regulator springs are placed. The main valve is balanced, and is provided with a flat seat and leather face. The upper part of this valve is provided with a cup leather packing and acts like a piston in the operating chamber. The pilot-valve is balanced against the delivery pressure by the regulator springs. The top of the spring chamber can be revolved by means of an operating handle attached to it. As it is provided with a coarse, square thread-screw, less than one revolution is sufficient to give the full range of pressures on the springs from full open to closed. By means of stops at the top and bottom of the stem, the pilot-valve is held positively in its two extreme portions, independent of the springs.

The full hydrant pressure is admitted to the operating chamber of the main valve through a small tube, which projects below the seat of the valve, in order to keep the entrance clear of the varying velocity near the valve-seat. When the pilot-valve is open, the water which flows through the small tube into the operating chamber wastes through the hose opening. This causes the pressure on the main valve to be unbalanced and this valve is, therefore, opened. When the pilot-valve is closed, the full hydrant pressure acts upon the operating piston and as its area is a little larger than that of the main valve, this valve is closed by the unbalanced pressure. Intermediate positions of the pilot-valve cause corresponding movements in the main valve, which affect the pressure at which the water is delivered.

Notches are provided on the outside of the regulator-valve for locking the operating handle at pressures of 50, 75, 100, 125, and 150 lbs., and at the full pressure up to 300 lbs. These pressures are marked on the cover of the valve at the corresponding notches. At each of the outlets a small *pet-cock* is provided, to which a pressure-gauge can be attached. By means of the arrangement described, the fireman can set the regulator-valve at the pressure he wishes to use, and is, thus, in this matter, independent of the pumping station.

The hydrant-head is provided with two handles, and can easily be lifted and attached to a hydrant by one man in 12 to 15 seconds. A latch on one side of the head holds it in place. The heads are carried on the hose-wagons or auto-trucks, and only a few are required for the whole fire department.

441. *Automobile hose-wagons*, capable of high speed, are housed at convenient places. Each wagon is equipped with a four-cylinder, four-cycle motor capable of developing a speed of 30 miles an hour through the streets of the city, with a load of 5000 lbs. Each wagon carries 2000 ft. of 3-in. hose and is equipped with one 2000-gal. Morse Invincible monitor nozzle and two 1100-gal. monitors.

442. *Quantity of Water Available.* — After a careful study of the situation, the National Board of Fire Underwriters recommended that in the congested district of Baltimore a total delivery of 15,000 gals. per min. should be available within an area not exceeding 100,000 sq. ft., at a pressure of not less than 200 lbs. per sq. in. at the hydrants. This requirement is met by four hydrants of the high-pressure system, each of which is capable of delivering 3800 gals. per minute with 200 lbs. pressure at the hydrant, through four lines of 3-in. rubber-lined hose, each 100 ft. long and provided with a $1\frac{3}{4}$-in. nozzle. At least eight hydrants are available in the high-pressure district for the units of area of 100,000 sq. ft., mentioned above.

443. *The Pumping Station.* — In the high-pressure fire system of New York, electric driven centrifugal pumps are used. In the Philadelphia system, gas-engine driven, geared, triplex plunger pumps were installed. The engineers who designed the Baltimore system considered *reliability* under the most adverse circumstances "to be most important requirement the pumps of the high-pressure system should comply with." They thought that economy in first cost, or in operating expense, was a less important consideration in such a pumping system than simplicity and certainty of operation. After a careful study of the New York and Philadelphia high-pressure fire systems, they decided to use steam pumps in Baltimore.

444. *Pumps.* — The plant is designed for four units of 4000 gals. per min. capacity, at a piston speed of 300 ft. per min., and making 50 r.p.m. Each unit consists of a horizontal, twin, simple, non-condensing crank and fly-wheel, plunger pumping engine, having the steam cylinders fitted with Corliss valve-gears. The engines are capable of working under a continuous pressure of 300 lbs. per sq. in. on the water ends, and were tested for a static pressure of 600 lbs. per sq. in.

An auxiliary pump is provided to take care of the leakage of the pipe-system, and to maintain a pressure of 150 lbs. per sq. in., as well as to provide for the first draft from the hydrants, before the main pumps get into action. This auxiliary is a horizontal, duplex, direct-acting, compound, non-condensing center-packed plunger pump, having a capacity of 1000 gals. per min. at a piston speed of 100 ft. per min.

Two air compressors are installed to maintain the air in the delivery great an obstruction to the flow, and it would be preferable to make a branch-connection, as described above. When several small corporation-cocks are used, instead of one large one, they should be placed, under ordinary circumstances, at least 15 inches apart, and the same distance from the hub and spigot of the main. This distance depends, of course, largely upon local conditions of soil, and is often fixed by the municipality or water company.

Instead of using several taps, the pipe may be reinforced by a *tapping-band*, called also a *service-clamp*, as shown in Fig. 249. With

FIG. 249. — Tapping-bands.

this arrangement, the hole drilled into the main need only have the inner diameter of the corporation-cock, the tap being screwed in this case into the band instead of into the pipe. A simple but efficient device for insuring a good joint between the main and the service-clamp is a molded lead-ring gasket (Fig. 250), which fits into a groove in the under side of the clamp. The gasket has extensions, or ears,

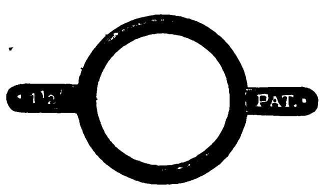


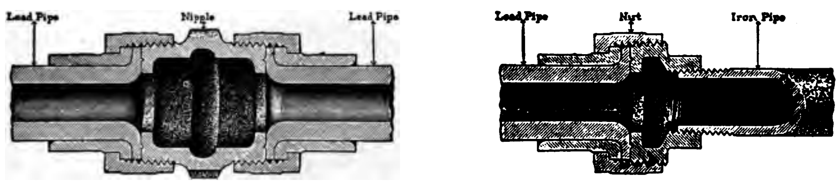
FIG. 250. — Moulded Lead Ring Gasket.

FIG. 251. — Lead Gooseneck.

which turn up at the outer edge of the clamp and hold it in place. Being of soft lead, the gasket fits itself to the irregularities of the pipe and makes a tight joint. Sometimes the gasket is made of rubber, or the joint is filled with cement.

Unless the service-pipe is made of lead, a lead gooseneck, about 2 feet long (Fig. 251), should be placed between the corporation-cock and the service-pipe, to enable this pipe to adapt itself to any settling that may occur. The top of the gooseneck must be kept deep enough in the ground to insure that the water in the pipe will not freeze. The gooseneck is connected to the corporation-cock and the service pipe by *wiped-joints*, *cup-joints*, or by means of *lead flange-joints*. The wiped joint requires skilled labor. The cup-joint, when properly made, involves less labor and material, and is perfectly satisfactory.

Flange-joints (Fig. 252) require only the use of a tool to spread and flatten the lead pipe in the joint, to form the gasket. Corporation- and curb-cocks of the lead flange pattern are, in many cases, preferable



LEAD PIPE LEAD AND INSIDE THREAD

FIG. 252. — Flange-joints.

to other types as they require no skilled labor and can be made in wet or muddy trenches that would make the use of molten lead dangerous. A laborer of ordinary intelligence, with a hammer, turn pin, and swedging tool, can make a lead flange joint in less time than a skilled man can make a wiped-joint. The lead flange-joint can, also, be readily disconnected by merely turning a nut with a monkey-wrench. The efficiency of the joint is not impaired by disconnecting and reconnecting it a number of times.

When multiple goosenecks are adopted, more flexibility will be given to the connection with the water main than when one gooseneck is used, as the lead pipes will have smaller diameters in the former case than in the latter. There is, therefore, less liability of breakage with several goosenecks in the connection than with one. All service-connections of whatever size should be protected by bricking or *housing in*, to prevent danger from the sudden settling of the earth. This inexpensive arrangement reduces the strains in the connections.

457. *Curb-cocks.* — The curb-cock is generally an ordinary stop-cock. Occasionally a stop-and-waste-cock is used for this purpose. Both types of curb-cocks are commonly used. Fig. 253 shows a curb-cock of a *round way pattern*, with bulging sides, which gives more room for the key, and makes it possible to give it a larger opening. The key or plug and the *T* handle are cast in one part. In Fig. 254 a more serviceable and dependable type of curb-cock is shown, the key or plug being protected by a patent cap which excludes dirt and all foreign substance from contact with the key. By this arrangement, it is impossible for dirt to get between the body of the cock and the key, and to cause the key to cut and leak. For ease of operation, the key is inverted. It yields readily to the pressure of the shut-off rod, but

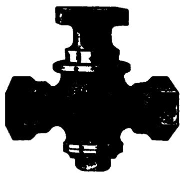


FIG. 253.

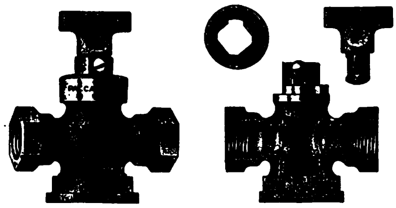


FIG. 254.

Curb-cocks.

is held to its bearings by pressure of the water through a by-pass at the bottom of the cock.

458. *Stop-and-waste cocks* are usually placed in the cellar or basements. Fig. 255 shows such a cock of the ground-key type, with patent cap to protect the key or plug. A small hole is drilled in one

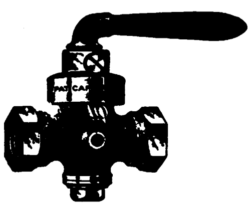


FIG. 255.

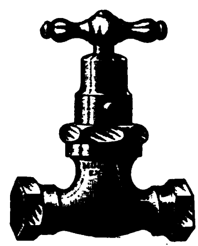


FIG. 256.

Stop-and-waste Cocks.

side of the key and reaches the opening or water-way. When the cock is shut off, the small hole in the key is opposite the opening of the pipe on the house side, while one of the main openings through the key is opposite the small waste-hole in the body of the cock. When the cock is closed against the service side, the supply is shut off, and the water left in the home-pipes drains off. Another kind of stop-and-waste cock, known as the compression type, is shown in Fig. 256. It has a double seated valve. When the supply seat rises, the waste seat closes, and vice versa. It is mechanically impossible for both to be open at the same time.

459. *Compositions for Corporation- and Curb-cocks.* — The following table gives compositions for corporation- and curb-cocks that have been adopted by various cities and manufacturers.

TABLE LVII. — CORPORATION- AND CURB-COCKS

Description	Copper, per cent	Tin, per cent	Zinc, per cent	Lead, per cent
Manufacturers' Standard Composition No. 1	85	4	7	4
Manufacturers' Standard Composition No. 2	87+2*	9		2
New York City	87±2	$5\frac{1}{2}\pm1\frac{1}{2}$	$6\pm1\frac{1}{2}$	$1\frac{1}{2}\pm1$
Providence, R. I.	80	6	3	2
Syracuse, N. Y.	88	6	3	3

* Silica copper.

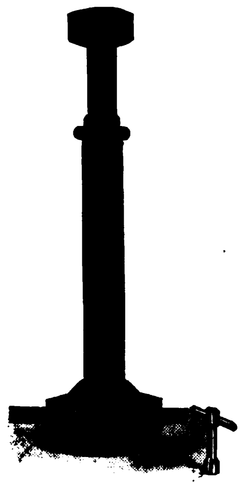
In soil that is alkaline, or contains salts, zinc should generally be omitted in the composition, as it causes the corrosion of the composition in such soil. This difficulty may be overcome by using the composition given above as the Manufacturers' Standard Composition No. 2, in which silica copper is used.

The keys or plugs should be lubricated with plumbago and Albany grease or vaseline. Beeswax and tallow should not be used, as they will get hard and make it difficult to open or close the valve.

460. *Service-boxes* are used to protect the curb-cocks. They should be made so as to prevent dirt and rubbish from getting into the curb-cocks, and should have covers which can only be removed by means of special keys, in order to prevent unauthorized persons from tampering with the boxes. As the distance from the curb-cocks to the sidewalk is variable, the boxes should be adjustable for different heights.

The usual kind of service-box is of the telescopic pattern, and consists of three parts: a base or lower section, an upper section, and a cover. The lower section is enlarged at the bottom so as to enclose the curb-cock. The upper section either slides over the lower one, or is connected to it by a threaded joint. In cold climates an objection to the latter arrangement is that the frost may lift the whole box, instead of only the upper section, and that it may be difficult to drive it back again. The covers are usually circular, except for boxes set in brick walks, which should have square or rectangular covers. The cover may be secured to the box by means of a brass screw bolt with a triangular or pentagonal head. In some service-boxes, a special casting is added at the bottom of the box, to keep the stop-cock in the center of the box and the tee-head in a vertical position. Some boxes are provided with a short rod, which is attached to the cock and held in the center of the box by lugs on the interior of the casing. This arrangement is useful in localities where the ground water rises in a soft material, and often fills the bottom of the base with mud and water.

Fig. 257 shows one of the five types of *perfect curb-boxes* made by the S. E. T. Valve and Hydrant Company of New York. It meets all of the general requirements, mentioned above, and has, in addition, a second cover, which protects the box, if the upper cover should be broken. Both covers lock without bolts.

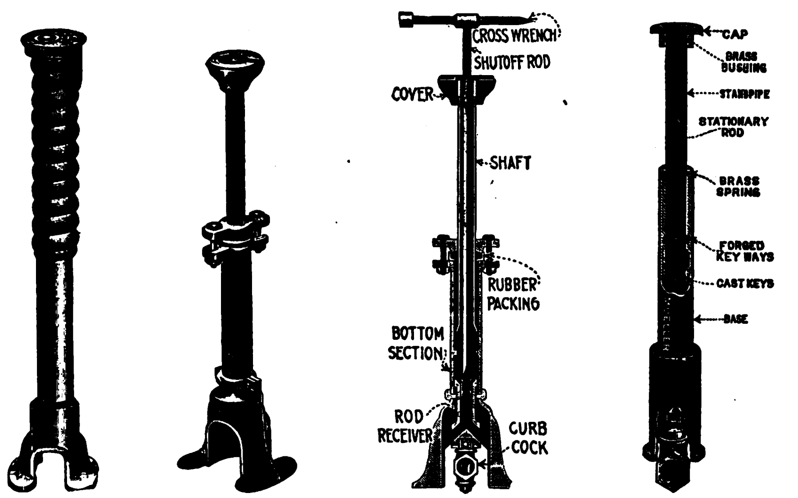


S. E. T.

Fig. 257.

Fig. 258 shows some other types of service-boxes. *The Buffalo service-box*, named after the place where it was first made, is substantial and does not get easily out of order. The box can be adjusted to the depth of the trench by screwing or unscrewing the top section.

The Quin-See service-box is light in weight, easy to handle and possesses many desirable features. The cover is made of cast iron, in one piece. In its center is set an iron screw in a brass bushing, to prevent rusting. When the iron screw is removed, a shut-off rod, up to $\frac{7}{8}$ in. in diameter, can be inserted in the box. The box is adjusted by a rubber packing gland, and is held at the desired length by its rubber packing, which can be made as tight as desired by means of the gland-bolts. A flange on its lower end prevents the shaft from being pulled through the gland. A special casting is placed over the corporation-cock, to receive the lower end of the shut-off rod.



BUFFALO QUIN-SEE MUELLER

Fig. 258. — Service-boxes.

The box can be extended, when necessary, by replacing the shaft, which is a piece of 1- or $1\frac{1}{4}$-in. pipe, by a piece of the desired length.

Fig. 258 shows another practical design for a service-box which is manufactured by the Hays Mfg. Co. of Erie, N. Y., and the H. Mueller Mfg. Co. of New York City.

461. Service-pipes. — The kinds of pipe which have been used for service connections are: lead, lead lined with tin; wrought iron, protected by a coating of asphalt and tar, zinc, lead, tin, enamel, or cement; and cast iron coated with asphalt and tar. In selecting the kind of service-pipe to be used in any particular case, the points to be considered are: (1) The chemical action the water may have on the pipe; (2) the cost of laying and maintaining the pipe; (3) its durability.

In most cases lead pipes will be found to be the best that can be used for services up to about $1\frac{1}{2}$ inches in diameter. While more expensive than wrought iron pipes, they are more durable and can be laid more cheaply, as they can easily be turned around obstructions that may occur in the pipe-trench and have a less number of joints. Considerable prejudice exists against the use of lead pipes, as they cause, in some cases, lead poisoning. This may occur, when soft water is stored in lead-lined tanks, or where water from springs or wells is conveyed for several hundred feet to the consumers through lead pipes. The poisoning by the lead is especially apt to occur, if the tank or pipes are alternately exposed to the air and the water.

462. *Lead Poisoning.* — The subject of lead poisoning caused by service-pipes has been carefully investigated by the Massachusetts State Board of Health, which arrived at the following conclusions:*

1. Nearly all serious cases of lead poisoning in Massachusetts, caused by the use of public water supply, have occurred in cities using ground water.

2. The greater the hardness of the water, as compared with its free carbonic acid, the less effect will the carbonic acid have upon the lead.

3. Surface water acts also, however, on lead pipe, but dissolves only a small amount of lead. If this water is distributed through lead pipe it may become injurious to health.

4. A water which has ordinarily only a slight action on lead pipe may, by some change of condition, take up a much greater quantity of lead than under ordinary circumstances, and a change in the source of the supply of a community has somet mes been followed by a material increase in the action of the water upon lead service-pipes.

* Reports of Massachusetts State Board of Health, 1898, p. xxxii; 1900, p. 488; and 1901, pp. xxxi and xxxii.

5. An occasional use of water containing a little lead may not be injurious, while the continued use would be, as lead is a cumulative poison. The exact quantity of lead that will be injurious depends upon the person, but it is known that the continued use of water containing as little as 0.05 of a part for 100,000 (about $\frac{1}{33}$ grain per gallon) has caused serious injury to health.

Lead poisoning is not apt to occur with most waters, if the lead service-pipe is short and the water remains in it only a short time — say, never longer than twelve hours. The water of Ancient Rome was distributed by lead pipes, and in most large modern cities in America the usual service-pipes for domestic supply are made of lead. With surface water, such as used in Boston, the lead service-pipes are soon coated on the inside with a brown layer of organic matter, under which a coating of carbonate of lead occurs. After these layers have been formed, the water has no further appreciable effect on the lead pipe.* It is a good precaution to waste the water that has stood in a lead or lead-lined pipe over night.

463. *Tin-lined lead pipes* have been used to some extent in Providence, R. I., and in other cities. While the tin coating protects the lead at first, a chemical or galvanic action is said to occur soon between the two metals, and tends to destroy one or the other. The tin lining is apt to be broken at the joints. The advantage of using these pipes is more than offset by the objections just mentioned, and by the higher price.

464. *Wrought Iron Pipes.* — Service-connections larger than 1½ inches in diameter are usually made with wrought iron pipes. Galvanized pipe has, probably, been used more extensively for this purpose than any other kind of wrought iron pipe. These pipes are made by giving them a bath in acid, and then dipping them in melted zinc. The coating thus formed is not always perfect, but, even when it is, the zinc becomes corroded in time. Prof. W. R. Nichols † says about this kind of pipe: "When the pipes are exposed to the action of the water, corrosion begins at once. At first, this action is on the zinc alone, provided the original iron was free from rust, and the treatment of the zinc thorough, but, after a time, the zinc which remains will cease to protect the iron and iron rust will begin to form. As regards this action, it is simply a question of time."

According to W. H. Richards,‡ a ½-inch galvanized pipe will clog in from 5 to 15 years, except with special waters, when it may last

* *The Sanitary Engineer*, December 6, 1883, p. 13.

† "Water Supply," by Prof. W. R. Nichols, p. 214.

‡ Jour. New England Water Works Assoc., June 19, 1884.

longer. After fifteen years' trial, the use of galvanized service-pipes was abandoned in Middletown, Conn., in 1884. As regards its effect on the health of the consumers, zinc pipes are more dangerous than lead pipes. In ordinary cases, however, no bad effect will result if the service-pipe is of ordinary length and is kept full of water.

Wrought iron pipes, coated with a preparation of tar and asphalt by some secret process, have been used to a limited extent for service-pipes. Enameled pipes, coated at high temperature by some patented process, have also been used for this purpose. A liquid enamel is sold with these pipes for coating exposed ends and special castings. As the coating is, however, applied, in this case, without any heat, it is doubtful whether it will furnish more than a temporary protection. Other coatings of tin, rubber, etc., have been tried for wrought iron pipes but have not proved satisfactory. Wrought iron service-pipes, lined with amalgamated lead (Fig. 259 *), are also made.

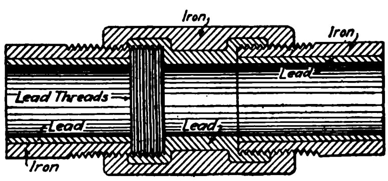


FIG. 259. — Lead-lined Iron Pipe.

465. *Cement-lined service-pipes* have been used in several cities and towns, especially in New England, for about thirty years. In New London, Connecticut, such pipes, having an inner diameter of $\frac{3}{4}$, 1, and $1\frac{1}{4}$ inches are lined in the following manner.† Portland cement with a small proportion of Rosendale cement is mixed with sand in the proportion of one to one. Originally only Portland cement was used, but the addition of a small amount of Rosendale cement was found to give better results. After being properly mixed, the cement-mortar is forced into the pipe by a special press. Two cones having the diameter of the finished bore are then drawn through the pipe, pressing the mortar to the side. The pipes, after being lined in this manner, are laid carefully aside, until the cement has set, when liquid grout is poured through the pipes to fill any holes that may exist in the cement lining. By this process, a 1-inch pipe is made into a $\frac{3}{4}$-inch cement-lined pipe, at a cost of about two cents per lineal foot. This kind of pipe is, next to lead, the best kind that can be used for service-pipes.

A serious objection to all kinds of coated wrought iron pipes is the fact that they are apt to rust where they are cut, the rust accumulating in the fittings and special castings. Brass cocks and fittings appear to have an affinity for rust, and are often completely obstructed in this manner.

* Manufactured by the Lead-lined Iron Pipe Co., Wakefield, Mass.

† Jour. New England Water Works Assoc., June 19, 1884.

CHAPTER XXIII

CLEANING AQUEDUCTS AND WATER MAINS

466. Vegetable and animal growths occur for a distance of one or two miles from the inlet in all aqueducts and pipe-lines that convey surface water. Beyond this distance the growths are less extensive. The principal growth is generally Polyzoa, which hold diatoms and other organisms, together with much amorphous matter. When the aqueduct is empty, the coating on its inner surface appears as dark brown slime, not over $\frac{1}{16}$ inch thick, but as soon as water fills the conduit again, tentacles from these growths float in the water and extend about $\frac{1}{4}$ inch from the surface. The capacity of an ordinary masonry aqueduct is reduced in this manner about 10–14 per cent in 7 or 8 months, but most of its original capacity is recovered, when the slimy coating has been removed.

In the brick-lined Cochituate aqueduct of Boston, Mass., growths of spongilla occur, if deposits of slime and dirt are allowed to accumulate. In course of a year or two, this growth becomes hard, and its projections form a serious impediment to the flow of the water.

Concrete conduits and cement-lined pipe appear to be less subjected to slimy coatings than brick-lined conduits. In twenty miles of cement-lined pipe in northern New Jersey, no growth or deposit was found, when the pipe was taken up.* The same condition was found in 1911 in the concrete aqueduct of Jersey City, which had been 8 years in service.

Vegetable and animal growths thrive also on the sides of open canals,† and reduce their discharging capacity considerably.

467. Cleaning Aqueducts. — Whenever possible, aqueducts should be inspected and cleaned at least once each year. In the Boston aqueducts the slimy coating on the sides and arch is removed once or twice per annum by means of water and hand brooms, or by water under a pressure of about 200 lbs. per square inch which is discharged by a gasoline power-sprayer against the inner surface of the conduit. The latter method is said to be the more economical of the two. The bottom of the aqueduct is cleaned by means of push-brooms, operated by hand.

* "Water Works Handbook," by Flinn, Weston, and Bogert, p. 291.

† Trans. Am. Soc. C. E., 1911, LXXI, p. 186.

468. Mechanical Cleaning of Water Mains. — As explained on page 461, the capacity of most water mains is gradually reduced by tuberculation, the degree to which this takes place depending upon the character of the water. Various machines have been used abroad for cleaning the water mains, but it is only in recent years that such contrivances have been used in America. Considerable experience is necessary in the design and construction of mechanical cleaners and in using them. The cleaning of water mains should be given out to contractors who have had special experience in this class of work.

469. *Pipe Cleaning Machines.* — Two types of pipe cleaning machines are used in the United States. The machine shown in Fig.

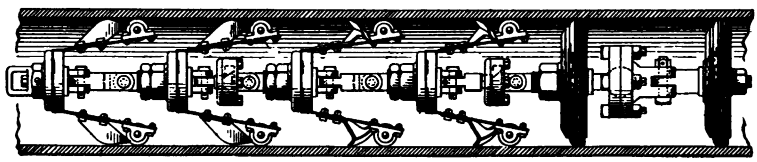


FIG. 260. — Hill-Hodgman Pipe Cleaning Machine.

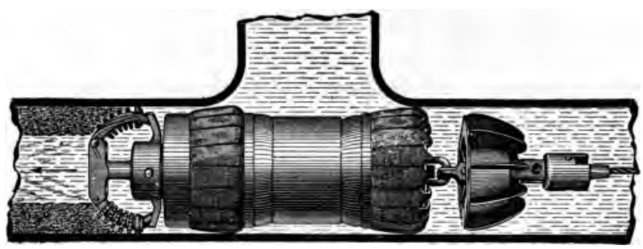


FIG. 260*a*. — Turbine Pipe Cleaning Machine.

260 is the Hill-Hodgman water propelled machine, and that shown in Fig. 260*a* is a turbine type of cleaner, patented by a German.*

The former machine is composed of a series of heads, flexibly connected, and carrying resilient springs, to which are fastened cutting and scraping members that conform to the inner surface of the pipe. The springs are provided with wheels, which effectually prevent the abrading of the tar coating of the pipe by the scrapers. The machine is propelled through the water main by means of a double piston in the rear, which is so constructed that the flow is under the control of the operator. On account of the practice in this country of allowing corporation-cocks to project into a main, it is necessary to so design cleaning machines that they will not injure or disturb the

* The United States patents for both of these machines are owned by the National Water Main Cleaning Co., New York.

corporation-cocks, while thoroughly removing the incrustation about them. This machine is designed to operate in this manner.

The turbine type of cleaner consists of a cutting head which is rapidly revolved in the pipe by means of a water turbine, attached to the rear of the cutter-head. In order to secure the full effect of the water passing through the main, flexible cups are provided as shown, and these conform to the inner surface of the main. The rate of travel of this machine is governed by means of a cable placed at the rear of the machine. When systematic cleaning of water mains is to be done, it is desirable to install special fittings called *hatch-boxes.* They should be placed in the mains at convenient intervals, in manholes, if possible, so that access may be had to them without having to dig or excavate in the street to the pipe. Fig. 261 shows the *Empire Hatch-Box,* which is caulked into the main and becomes part of it. The cover is bolted and leaded to the box, and the box is so designed that the cleaning apparatus can be inserted in the main without difficulty. With the Hill-Hodgman machine, this hatch-box

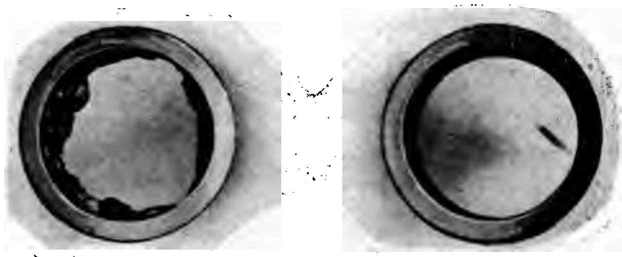
(PATENTED)

FIG. 261. — Empire Hatch-box.

suffices, but with the turbine type of machine, involving the use of a cable, it is necessary to provide the cover of the hatch-box with a stuffing-box and pulley for feeding the cable into the main. In particularly bad cases, when conditions do not warrant the use of a water-propelled machine, a special machine, drawn by means of a cable, is used. In this case a preliminary operation is necessary, in order to get the cable through the main.

470. *The process of cleaning mains,* as carried on in the United States, is performed as follows:

The main is opened up in two places, where it has been exposed by excavating down to it. The distances between these cuts in the main vary according to its diameter, the pressure maintained, and the nature of the incrustation. It will, therefore, be seen that experience is necessary in order to decide upon these distances, so that the work can be carried on economically. Where there is sufficient water pressure, and the condition of the interior of the main warrants it, water-propelled machines are used. If hatch-boxes are installed, it is only necessary to remove the cover, insert the machine, replace the cover, and turn on the proper amount of water to give the machine the proper rate of travel through the main. When water pressure cannot be used, a cable is passed through the main by means of a special carrier, and after it has passed through the main, a machine is attached to it and inserted into the pipe-line through the hatch-box. The cover is then replaced, the proper amount of water is turned on, and the machine is pulled through the water main by means of a windlass. Whether the pipe cleaner is propelled by water or drawn by cable, a certain quantity of water is allowed to pass the machine in order to carry ahead of it the dirt and incrustation which has been cut and ground up by the machine. The water with this sediment and cleaners is carried to the surface of the street through a *riser-pipe.* After



BEFORE CLEANING AFTER CLEANING

FIG. 262. — Effects of Cleaning Water Mains.

the machine has passed through the main, the water is shut off, the riser-pipe is removed and the main at that point is reconnected.

Fig. 262 illustrates the effects of cleaning water mains by means of a machine.

Many unexpected difficulties are encountered in cleaning water mains and cause delay and trouble. In extremely bad cases, small mains have to be opened at points as close together as 100 feet. Owing to the carelessness in the original laying of the pipes, lumps of lead may have entered the pipes at a joint which may be large enough to prevent the passage of the cleaner, and may necessitate an extra cut. Water departments are often not sure of the location of the pipe and of the valves and special fittings. This may cause loss of time and extra excavation. Lack of knowledge of the condition and position of valves often makes a complete shut-off impossible, until the necessary information is acquired at considerable expense and trouble.

CHAPTER XXIV

THAWING FROZEN PIPES AND HYDRANTS

471. Thawing Frozen Mains. — Where frost occurs, the water mains should be laid below the deepest frost line, to prevent the water in them from freezing in cold weather. The depth to which the pipes should be laid varies from 4 to 7 feet in different parts of the United States. The danger of the pipes becoming frozen is especially great *at dead-ends* where there is no circulation. Pipes covered with a rock fill and surrounded, more or less, by air are less apt to freeze than pipes laid in wet ground. When a pipe has become frozen, it can be thawed either by steam or by electricity.

472. *Steam Thawing.* — The manner of thawing a frozen pipe by steam adopted by G. P. Pruett, City Engineer of Miles City, Montana, in the severe winter of 1915–16 was as follows: *

"Holes were first driven down to the pipe, about 20 ft. apart, by means of 1-in. drills, which were pounded down with a sledge hammer. Steam was shot into these holes until the frost was drawn, and then the pipe was uncovered at these holes in sections of about 5 ft. Short pieces of pipe were worked along the top of the water pipe and thoroughly steamed from one hole to the next. The main was tapped to locate the frozen section, which was about 100 ft. long, and the steaming process was continued, from hole to hole, until the main was opened. The pipe did not break and gave no further trouble from freezing, although the frost was below the main, before it was steamed."

473. *Electric Thawing.* — In March, 1912, the New York Edison Company † thawed out by electricity a 6-in. cast iron main, 1700 ft. long, which supplies a large hospital on Brother Island, in New York City. When the Company began operations, the pipe had been frozen for a month, all attempts to thaw it out by steam having failed. Four, and later six, transformers for 2000 to 200 volts were used. The pipe was cut on both shores and a current of 800 amperes was turned on, a water pressure of 80 lbs. being maintained on the island end of the pipe by electric pumping, to assist in forcing out the ice. The current acting on the pipe was maintained for five days, being gradually increased to 1800 amperes at about 400 volts. On the fifth day the

* *Engineering and Contracting*, 1916, XLV, p. 437.

† *Edison Monthly*, April, 1912.

water began gradually to flow, and the pipe was soon free of ice. As the cold salt water absorbed much of the heat from the pipe, it required thirty-six times as much heat to thaw out the pipe as would be required to melt an equal quantity of ice on land.

474. Frozen hydrants cause much trouble in cold climates. The trouble is generally caused by improper drainage, or by the branch pipe — which forms a dead end — not being laid deep enough. The hydrants are usually thawed out by means of a portable boiler, hooked directly onto the nozzle, with a cap, tapped and connected with the steam hose. A steam pressure of about 50 lbs. per square inch will usually do the work. The hydrant is kept closed and the drain valve is opened, so as to heat the hydrant and, also, the surrounding ground. In ordinary cases the hydrant may be thawed out in ten minutes at a trifling cost. If a rainstorm occurs, just before the freezing weather begins, the frost will penetrate deeper than it generally does, as the ground will be full of water. Hydrants or their branch-pipes, which ordinarily caused no trouble, may under such circumstances become frozen.

475. Thawing Frozen Service-pipes by Electricity. — If service-pipes are not laid below the deepest frost-line, the water in them may freeze in winter, thus cutting off the supply. Formerly the pipes were thawed by steam, after they had been partially or entirely exposed, but this can be done more expeditiously and economically by means of electricity. In the severe winter of 1915–16 about 10 per cent of the service-pipes in Miles City, Montana, were frozen between the mains and the basements of the houses, the trouble occurring principally at the goosenecks and at the curb-boxes. The manner in which the pipes were thawed is described by G. C. Pruett, the City Engineer of Miles City, as follows:*

"The frost was so deep and so hard that an attempt to expose the pipe by digging down to it meant considerable delay in re-establishing the service. For that reason the city purchased the necessary equipment, which consisted of one 1100 to 50-volt transformer, cable, meter switches, choke-coil, etc., and constructed an electric thawing apparatus. The whole equipment was placed on a trailer and pulled by a Ford roadster. In thawing, connection was made with the electric mains, the current was stepped down to 50 volts, and connections were made to the house water system and the nearest fire hydrant. The amount of current was controlled by the choke-coil and checked by an ammeter in the thawing circuit. For ¾-in. services the current was not allowed to exceed 150 amperes, and at this rate a service,

* *Engineering and Contracting*, 1916, XLV, p. 437.

which had not been frozen more than a day, was thawed in about 20 or 30 minutes, although in some cases one to three hours' time was required. With that amount of current, it was impossible to thaw frozen goosenecks. These were thawed by digging them up or by using a portable steam rig.

"The reason for limiting the current to 150 amperes was because by trial this was found to be the maximum which could be successfully used, without danger of causing the pipe or gooseneck to burst. Heavier currents melted the wiping between the lead and brass or heated the pipe so quickly as to cause it to break. After a little experimenting, only one connection in about 100 thawed was broken.

"The charge made for thawing was $5 per hour or connection and $5 for each hour after the first, *i.e.*, the minimum charge was $5. This only covered the rent of the rig and three men to handle it, who were employees of the City Light Department. In addition to this, the consumer was required to have a plumber, to take care of any pipe work and to look after leaks, in case they should develop."

CHAPTER XXV

LEAKAGE FROM AQUEDUCTS, MAINS, AND SERVICE-PIPES

476. Leakage from Aqueducts. — A certain amount of leakage, outward or inward, occurs in all masonry conduits. If the ground water is of good quality and under a greater head than the water in the aqueduct, it should be admitted freely into the conduit through *weepers* — small openings provided for this purpose at regular intervals, — as it increases the supply. The quantity of ground water that enters the first 25 miles of the new Croton aqueduct (p. 277) is estimated roughly at about 4,000,000 gallons per day. In 7 miles of this aqueduct, which is under a pressure of about 130 ft., the outward leakage amounts to about 32,100 gallons per day per mile. If the ground water is of bad quality, it must be excluded from an aqueduct, by making the masonry as water-tight as possible and by lining it at places, where the ground water is under great pressure, with metal — usually cast iron or steel.

The outward leakage from well constructed gravity conduits is not apt to be very large and diminishes gradually by silting. In pressure-aqueducts, however, the outward leakage may be large, and every possible means should be taken to reduce it to a minimum. This is generally accomplished by forcing grout under pressure into all fissures and seams from which water flows into the aqueduct. In the pressure tunnel of the Catskill aqueduct (p. 306), which was driven 1100 ft. under the Hudson River, leaks into the tunnel were stopped by grouting under a pressure of 300 lbs. per square inch. In the city tunnel of the same aqueduct, which was driven 200 to 750 ft. below the surface, a copper lining was placed inside of the concrete lining for a distance of about 1500 ft. At this place the aqueduct is circular with a diameter of 12 ft.

In the Jersey City conduit, which is lined with concrete and has an inner diameter of 8.5 ft., the inward leakage on one of the sections, of which 1600 ft. was in tunnel in shale and sandstone, amounted during the construction to about 154,000 gals. per day per mile.

In the land tunnel of the Cincinnati Water Works,* the leakage inward was at first at the rate of 3400 gals. per day per mile, but this

* "Water Works Handbook," by Flinn, Weston, and Bogert, p. 298.

increased in eight months to about 5800 gallons per day per mile. The maximum leakage, concentrated into 1100 ft. of tunnel, amounted to a rate of 117,500 gals. per day, per mile under a head estimated at about 75 ft. The maximum outward leakage under a head of 34 ft. was 14,000 gallons per day for the whole tunnel, which had a length of 1500 ft. and a diameter of 10 ft., and for the wet stretch it amounted to 290,000 gals. per day per mile. This tunnel was driven through very tight rock, only 5 per cent of which was wet.

477. Leaky Mains and Service-pipes. — A large part of *the water unaccounted for* in water works is due to leaky mains and service-pipes. Emil Kuichling estimated the probable loss per second by leakage in a well constructed pipe system as follows: * One drop from each pipe-joint, 5 drops from each hydrant or gate-valve, and 3 drops from each service-pipe. Estimating a leakage of 1 drop per second equal to 3 gallons per day, Kuichling figured the average daily loss by leakage at about 2500 to 3000 gallons per mile. This estimate does not, however, take into account the fact that more leakage is to be expected from large pipe-joints than from small ones.

G. E. Bradbury made some tests, to ascertain the loss of water in the mains of Akron, Ohio, and collected data bearing upon this subject for other towns.† He recommended that this source of loss should be given per mile of water main per inch of diameter, or per lineal foot of lead joints, instead of being given as a percentage of the total supply or as a leakage per capita. In the specifications for laying additional pipes for improving the Akron Water Works, the permissible daily leakage was limited to 200 gals. per inch-mile of pipe, which was about equivalent to 1.6 gals. per foot of lead joint. The results obtained by careful tests made of these pipes, after they were laid, are given by Mr. Bradbury as follows:‡

* Trans. Am. Soc. C. E., 1897, XXXVIII, p. 18.

† Proc. Ohio Engineering Soc., Jan. 25, 1912, also *Eng. Record*, Feb. 3, 1912, p. 138.

‡ Jour. New England Water Works Assoc., 1914, XXVIII, p. 319, also *Eng. News*, Oct. 8, 1914, p. 725.

TABLE LVIII. — TESTS OF PIPES LAID IN AKRON, OHIO

Pressure from 66 to 152 lbs. per sq. in.

Size of pipe, inches	Length, feet	Daily leakage per inch-mile, gallons	Number of tests
1. Laid by contract			
4	717	23	1
6	31,066	66	34
8	6,882	42	6
10	5,123	81	5
12	9,704	102	8
16	8,792	135	11
20	8,389	69	10
24	3,358	69	3
30	14,445	82	8
	88,476 =16.76 miles	83.4 average	86
2. Laid by superintendent of water works			
6	37,524	59	32
8	6,509	63	5
10	2,850	133	1
	46,883 =8.9 miles	61.7 average	38

The figures given are probably high, as they include all leakage that may have occurred through gates, as well as actual loss.

Based upon the above tests, Mr. Bradbury recommends that all pipes be tested by hydraulic pressure before being covered, and that the allowable leakage for new cast iron water pipes be limited to an average of 100 gallons per mile per inch of diameter for each complete contract or district, with a maximum limit of 200 gals. per mile per inch of diameter not to be exceeded in any single tests.

In connection with the water purification works of Columbus, Ohio, about 19,000 lineal ft. of 24, 30, and 36 ins. cast iron pipes were laid. According to the specifications, the pipes were to be tested, after being laid, under a pressure of 110 lbs. per square inch, and the allowable leakage was limited to the following quantities:

Diameter of pipe	Allowable leakage per lineal foot of pipe per hours, gals.
20	0.08
24	0.10
36	0.15

The results of the tests for leakage made on these pipes were as follows:

TABLE LIX. — LEAKAGE TESTS ON WATER MAINS AT COLUMBUS, OHIO *

Diameter of pipe, inches	Length of pipe, feet	Joints		Total length of joints, feet
		Number	Length per joint, feet	
36	2046	228	9.425	2149
24	81	19	6.283	119
30	54	11	5.236	58
	2181			2326
36	4195	360	9.425	3393
36	4275	379	9.425	3572
36	8470	739	9.425	6963

* *Eng. Record*, April 20, 1912, p. 432.

TABLE LX. — MEASURED LEAKAGE IN GALLONS

Total per hour	Gallons per hour per lineal foot of joint	Gallons per hour per lineal foot of pipe	Gallons per 24 hours per mile of pipe	Gallons per 24 hours per inch diameter per mile of pipe	Pressure in pounds per square inch
330	0.142	....	....	....	117
447	0.132	0.107	13,500	375	110
571	0.160	0.134	16,900	469	110
1018	0.146	0.120	15,200	422	110

The average measured leakage on two of the 36-in. pipe-lines, under 110 lbs. pressure, was 422 gals. per 24 hours per inch diameter per mile of pipe, whereas the specifications permitted a corresponding maximum leakage of 528 gallons.

Careful tests have been made for a number of years of the leakage in the pipe system of Washington, D. C. The results of these tests were as follows:

TABLE LXI. — UNDERGROUND LEAKAGE IN PIPE SYSTEM OF WASHINGTON, D. C.

Year	Total leakage, gallons per day	Leakage from joints in mains, gallons per day	Per cent of total leakage found in joints
1908	4,243,900	1,013,900	24.0
1909	6,657,635	1,345,620	20.2
1910	6,364,000	1,034,000	16.2
1911	6,921,916	2,562,461	37.1
1912	5,115,320	746,305	14.6
1913	4,196,070	962,310	23.0
		Average	23

Dexter Brackett gives the following leakage in the pipe system of Boston and of the Metropolitan Water Works, according to tests made in 1894–1900, soon after the pipes were laid.

TABLE LXII. — LEAKAGE IN WATER PIPES OF BOSTON AND OF THE METROPOLITAN WATER WORKS, 1894–1900

Diameter of pipe, inches	Sum of lengths tested		Average leakage per lin. ft. of pipe, gallons per 24 hours	Average leakage per lin. ft. of lead joint, gallons per 24 hours
	Feet	Miles		
48	116,563	22.0	3.7	3.16
42	11,744	2.2	2.5	2.43
36	25,663	4.9	2.9	3.19
30	7,287	1.4	0.6	0.81
24	20,553	3.9	2.1	3.44
20	40,231	7.6	3.6	7.00
16	49,903	9.4	1.9	4.65

The total length of the different sizes tested was 271,944 ft. and the average leakage per linear foot of lead joint per 24 hours was 3 gals.

CHAPTER XXVI

DURABILITY OF WOODEN AND METAL PIPES

478. Maintenance of Wood Pipes. — Lines of wood pipe should be frequently inspected, to discover any leaks that may occur and to make the necessary repairs. If the leakage is allowed to continue and the water carries sand or grit, it may cut through one or more of the bands and necessitate extensive repairs. Small leaks in the joints of the pipe are usually stopped by driving in wooden wedges. When the leaks occur in the wooden couplings of machine-banded pipe, the wedges should be driven into the staves of the coupling sleeve and not between them and the pipe.

The maintenance of a continuous stave-pipe is not difficult. Occasionally staves or portions of them have to be replaced. This can be done by removing a few bands, putting in new staves or parts of staves, where required, and rebanding the pipe at the place in question. Stave-pipes are sometimes injured by roots of trees, especially of willows, growing to the pipe. Such trees, etc., should be removed. If the pipe is exposed and supported by cradles, the latter must be kept in good condition, to prevent the pipe from settling. Weeds must not be allowed to grow near the pipe as they may catch fire and injure or destroy the pipe.

479. Durability of Wood-stave Pipe. — If made of good material, covered with compact, water-tight earth that is free of alkali or organic matter, and kept practically constantly saturated with water under a head of 25 ft. or more, wood-stave pipe will last 40 to 50 years. In porous or alkaline soil, or where the wood is not kept saturated, either on account of a low head, or because the pipe is empty for a considerable time, the life of the pipe may not be more than 10 to 20 years. Under these conditions the bands will give out first, and by renewing them the life of the pipe may be prolonged. Exposed wood-stave pipes, resting on cradles and kept constantly saturated, will last on an average about 25 to 30 years, depending upon climatic conditions.

The average duration of wood pipe of various kinds, made from different kinds of wood, has recently been carefully investigated by D. C. Henny, Consulting Engineer of the U. S. Reclamation Service.*

* *Reclamation Record*, Aug., 1915, p. 354.

The conclusions at which he arrived, as regards the relative life of fir and redwood in pipe, are as follows:

"(*a*) Under favorable conditions of complete saturation, fir, well coated, may have the same life as redwood uncoated.

"(*b*) Either kind of pipe will have a longer life if well buried in tight soil than if exposed to the atmosphere. Such life may be very long, 30 years and over, if a high, steady pressure is maintained.

"(*c*) Either kind of pipe will have a longer life if exposed to the atmosphere than if buried in open soil, such as sand and gravel and volcanic ash, provided in hot and dry climate it is shaded from the sun.

"(*d*) Under questionable conditions, such as light pressure or partially filled pipe, fir, even if well coated, may have only from one-third to one-half the life of redwood.

"(*e*) Under light pressure the use of bastard staves in fir pipe should be avoided.

"(*f*) The use of wooden sleeves in connection with wire-wound pipe is objectionable and has caused endless trouble and expense. It is possible that the objection may be partially overcome by dipping ends of sleeves in creosote and by applying a heavy coating of tar to the ends of the sleeves. Saturation of sleeve wood will never be as perfect, however, as of the straight pipe, and full creosote treatment of the wood, or else some form of metal sleeve, either riveted iron or steel, heavily coated, or cast iron, will probably be well worth its extra cost.

"(*g*) If wooden sleeves are employed, they should be provided at least for sizes from 10 inches up with individual bands, to permit taking up leaks.

"Pitch seams do not occur in redwood. In fir they should be distinctly limited as to size, frequency, and depth. In respect to knots, there appears to be no reason for making any distinction between the two classes of wood. Small, sound knots, if not passing through the full thickness and not occurring close to edges or ends, might be permitted in either wood. Sap is objectionable and the higher cost in prohibiting it entirely, or of putting narrow limitations on it, is probably justified in both fir and redwood.

"Wooden pipe is not suitable in cases where it cannot be kept full and under pressure during periods of use. Coating cannot, under such conditions, be expected to afford protection against decay. Coating should be continuous and heavy, not less than one-sixteenth inch, to be fully effective, and should preferably consist of more than one individual coat of a mixture of asphaltum and tar, or of an application of gas-tar, followed by one or more applications of refined coal-tar. Little experience, however, can be quoted in support of all-tar coating.

"All tentative conclusions herein presented are necessarily based on the limited data which could be collected and will undoubtedly require amendment as further experience accumulates."

480. Durability of Cast Iron Pipe. — The life of a cast iron pipe depends upon the chemical composition of the iron of which it is made, the nature of the material with which it is surrounded, and upon the character of the water passing through the pipe. Soft waters, as a rule, will corrode cast iron more rapidly than hard waters. If placed to a sufficient depth in fresh water, cast iron will rust slowly; but in salt water it corrodes rapidly. According to Mr. Cavallier, director of the foundries at Pont-a-Mouson, France, small cast iron pipes, laid in 1685 to supply the fountains of Versailles, France, with water were found in 1901 to be in good condition.* The same authority reports that a cast iron intake-pipe, which was laid in the Seine at Paris in 1802, was found to be in good condition when it was taken up about a century later. The 36-in. mains, which were laid prior to 1842 to convey Croton water into the city of New York, are still in service and in good condition. These pipes were cast in Scotland, and were given a wash of lime as a protective coating. On the other hand, pipes laid near the river fronts of New York in made-ground, containing ashes, rubbish, etc., and exposed to sewage contamination, will barely last 20 years. As a general rule, the average life of good cast iron pipes, properly coated with asphalt and placed in good ground, may be taken as 75 to 100 years.

481. *Tuberculation.* — The corrosion caused on the inside of a pipe by the foreign substances contained in the water differs somewhat from the rusting that takes place on the outside of the pipe. Small knobs or nodules are caused by the chemical combination of the iron with one or more of the substances in the water. This action takes place sooner in uncoated pipes than in those covered with a suitable protective coating and, consequently, attacks coated pipes at points where the coating is either worn off or imperfect. Tuberculation is caused by acids, particularly carbonic acid, liberated by organisms present in the water, by free oxygen and alkalis; but neutral water has been found to have the same effect.† Soft waters, being generally acid, cause this corrosive action more rapidly than hard waters. For this reason surface waters generally cause more trouble from tubercula-

* Jour. New England Water Works Assoc., XVIII, 1904, p. 219.

† Paper on Tuberculation and the Flow of Water in Pipes by Nicholas S. Hill, Jr. M. Am. Soc. C. E., in Proc. Amer. Water Works Assoc., 1907, p. 371.

tion than ground waters. However, in Brooklyn, N. Y., Welch, West Va., and some other places ground water has caused much tuberculation, so that no fixed rule can thus far be laid down about this matter. In some cases highly alkaline waters form a preliminary deposit which protects the pipe, in a measure, from further action. After this incrustation has reached a certain thickness, its further progress is much retarded.

In pipes coated with asphalt and tar a very small defect in the coating is sufficient to allow a tubercle to start, and the iron at this spot becomes often so soft that it can be cut with a knife. A depression is thus made in which the tubercle is formed. The tubercles are sometimes as large as $1\frac{1}{2}$ to 2 ins. in diameter and $\frac{1}{2}$ to 1 in. thick.

Freeland Howe, Jr., chemist and bacteriologist, has advanced the following theory to account for tuberculation.* When the water and its contents are dissociated into their component ions, the ions become more active chemically. The weaker the solution is, the more complete the dissociation will be. It is well known that all natural waters contain salts in very weak solution and they would, therefore, be expected to be completely dissociated, or nearly so, and hence very active chemically. The action on the pipes containing the water depends, of course, upon the quality and the quantity of the ions. If this theory is correct, there is practically no water which will not corrode, to some extent, that part of the water main which can be reached on account of imperfect coating. Mr. Howe states that, in general, ground waters affect all pipes more than surface waters in some ways, but that in others they would have less effect on the ions, on account of the larger amount of salts which would form protective coatings. These protective coatings are formed by waters having their sources in such formations as limestone, shale, and other rocks, in which the salts of calcium and magnesium are dissolved by the water.

482. Corrosion of Wrought Iron and Steel.† — Wrought iron, when protected against the action of the elements, will last for centuries. Pieces of wrought iron, supposed to date from periods of 2000 to 3000 B.C., have been found in pyramids, tombs, ruins, etc., in Egypt, Assyria, Greece, etc. While some of these specimens went to pieces, when exposed to the air, there are a number of others, preserved in museums, that are in a remarkably good condition.

The oldest specimen of wrought iron is, probably, a piece about

* Jour. New England Water Works Assoc., 1908, XXII, p. 43.

† Paper by Emil Kuichling in Jour. New England Water Works Assoc., 1910, XXIV, p. 514.

$2 \times 4 \times \frac{1}{8}$ in. in size, in the British museum, that was discovered in 1838 in an inner mortar joint of the stonework of the great pyramid of Cheops at Gizeh, Egypt. It is supposed to have been part of a mason's tool.

A fragment of wrought iron was found under the base of the obelisk, which was moved in 1880 from Alexandria, Egypt, to Central Park, New York. On being analyzed it was found to contain the following percentages of different elements: Carbon, 0.521; silicon, 0.017; sulphur, 0.009; phosphorus, 0.0048; manganese, 0.116; nickel, cobalt, and copper, together, 0.161; lime and slag, 0.370; pure iron, 98.738. Its tensile strength was found to be 54,500 lbs. per square inch, with an elongation of 14 per cent. The age of this specimen is supposed to be about 2000 years.

While the specimens of iron, mentioned above, were doubtless well protected against corrosion, there are examples of pieces of iron lasting for centuries in places where one would expect them to have been destroyed by rust.

Schliemann found in his excavations of ancient Troy in Asia Minor and of Mycenæ in Greece, a number of wrought iron keys, knives, etc., dating from 1400 to 1500 B.C., which were still in a good state of preservation.

Wrought iron pipes $\frac{3}{4}$ in. and 1 in. in diameter have been laid in many cities in all kinds of soil, as service-pipes for distributing gas. Although these pipes are not protected, as a rule, by any preservative coating, they last for long periods of time. Emil Kuichling * mentions examples of such pipes in Rochester, N. Y., which had been in use for more than 55 years.

483. *The Rochester pipe conduit* (p. 324) consists of 24-in. cast iron mains for the first $15\frac{1}{2}$ miles from the intake. Next follow 3 miles of riveted wrought iron pipe, 24 ins. in diameter; and for the remaining 9.62 miles the conduit consists of a riveted pipe, 36 ins. in diameter, made of wrought iron plates, $\frac{3}{16}$ to $\frac{1}{4}$ in. thick, according to the pressure to be resisted. The pipes were protected against corrosion by a thick coating of refined Trinidad asphalt and coal-tar pitch, mixed in equal proportions and applied at a temperature of 320° F. Where the coating was injured by transportation, this was remedied in the field, as much as possible, by painting.

Although these pipes were laid in clayey soil, no signs of corrosion were noticed for the first 19 years. From 1894 to 1900 fifty-six small holes, none larger than $\frac{1}{4}$ in. in diameter, were discovered in the 9.62 miles of 36-in. pipe, where the thickness of metal was only $\frac{3}{16}$ in. The holes, caused by rusting, in the course of 36 years, occurred in about

* Jour. New England Water Works Assoc., 1910, XXIV, p. 526.

25 to 30 plates out of about 11,300. Several of the holes were occasionally in the same plate. They were all plugged with wood, without interrupting the flow in the pipe. No additional holes were reported in this length of pipe from 1900 to 1910. Very few rust holes occurred in the 2.93 miles of 24-in. riveted pipe.

In 1893–94 a second riveted pipe was laid to secure an additional supply of water for Rochester from Hemlock Pond (p. 324). It is 38 ins. in diameter and 26 miles long. The pipe was made of open-hearth steel plates, $\frac{1}{4}$ to $\frac{3}{8}$ in. thick. For the greater part of its length, the pipe is laid in stiff, red or blue clay. Three different kinds of coatings were used to protect the pipe against corrosion, *viz.:* California asphalt (maltha); a mixture of Trinidad asphalt and coal-tar pitch, similar to that used for the first pipe-line; and a "baked" or "Japan" coating, devised by Prof. A. H. Sabin. Although the steel used for this pipe-line was the best available, and all the work performed in making and laying the pipe was excellent, the pipe began to corrode badly in 1901, seven years after the completion of the work. In seven places, where the pipe was coated with California or Trinidad asphalt, perforations, caused by rusting, were found. Prof. F. L. Kortright, of Morgantown, W. Va., who made a careful study of the causes of the corrosion of this pipe in 1901 and 1902, reported that it was due to the action of mill scale, oxygen, and free carbonic acid in the steel, being liberated at points where the coating was defective, and that it was not caused by gases in the soil or ground water. He advised thorough scraping of the metal where corrosion occurred; and then the application of two coats of mixed red lead and graphite paint, followed by two additional coats of mineral rubber paint or Truscon special paint. Up to the end of 1907, the pipe was repaired in this manner at 205 places, where rust holes were discovered. With one exception, these holes all occurred in plates that were only $\frac{1}{4}$ in. thick. Two companies furnished the steel plates for this pipe-line, *viz.:* The Pennsylvania Steel Company and the Carnegie Steel Company. The plates of the former, which were protected by the Sabin Japan coating, resisted corrosion better than those of the latter which were coated with California or Trinidad asphalt. This may, however, not have been entirely due to the difference in coating but may have been caused partly by the fact that the Carnegie Steel contained much more manganese than the steel of the Pennsylvania Company. The presence of this element appears from present experience to facilitate the corrosion of steel.

484. *The Bedford pipe-conduit,* laid in 1896, is 48 ins. in diameter and 8.25 miles long. The pipes were made of steel plates, $\frac{5}{16}$ in. thick, coated with California asphalt. Since its completion this pipe has been carefully inspected, as trouble with electrolysis has been experienced in the distributing system. In 1901 some damage caused by electrolysis was discovered at a certain locality, but up to 1910 no rust holes were found. The interior of the pipe was examined in 1908 and 1909. This inspection showed that the asphalt coating had become very brittle and that many tubercles had been formed, especially at the field joints. This trouble was found to be greater at the bottom than upon the sides and top of the pipe. Beneath the tubercles pittings, many of them 0.075 to 0.095 in. in depth, occurred. Many large blisters, filled with water, were found in the coating, but under the water the metal was free of rust and pits.

In contrast with the cases mentioned above, some steel pipes may be cited which appear to have resisted corrosion successfully for a considerable period of time. The most notable example of this kind is the pipe conduit supplying the city of Newark, N. J.

485. *The Newark Pipe Conduit.* — In 1890–01 the East Jersey Water Company constructed a riveted steel pipe, 48 ins. in diameter and 21 miles long, to supply the city of Newark, N. J. The pipe was made of plates $\frac{1}{4}$ in. thick, coated with California asphalt (maltha). No leakage from corrosion has thus far been reported to have occurred in this pipe.

486. *The Coolgardie Pipe-line* (p. 338). — After having been in service for a few years, this pipe-line showed signs of corrosion. The greatest deterioration occurred where the pipe was buried in the ground. Sections lying above the surface showed comparatively slight damage.

An interesting account of the cost of maintaining and repairing the long Coolgardie pipe-line is given in the annual report for 1914–15 of P. V. O'Brien, engineer in charge of the Works.* According to Mr. O'Brien, the cost of maintaining the pipe-line from its completion to 1910 was as follows:

TABLE LXIII. — COST OF MAINTAINING THE COOLGARDIE PIPE-LINE

Year	Cost	Year	Cost
1902–3 } 1903–4 }	$83,500	1909–10	$83,000
1904–5	37,600	1910–11	61,200
1905–6	38,000	1911–12	83,000
1906–7	68,700	1912–13	111,000
1907–8	96,600	1913–14	192,600
1908–9	74,500	1914–15	248,000

* *Eng. Record*, Oct. 21, 1916, p. 501.

In 1914–15 about $154,000 was spent in uncovering about 80 miles of the pipe and in recoating it where the external corrosion was bad. Where the pipe was above ground, it was found to be in a good condition, but where it was laid underground, it was generally in a more or less corroded condition, except at a few places.

The corrosion on the outside of the pipe was due to rusting, pitting, or scaling. Where the trench was in iron, stone, gravel, or sand, the pipe, although covered, was in a good condition. In such places it was practically free from pitting or scaling, although freely covered with spots of rust under the decayed coating. This rusting was very slight and did not damage the pipe appreciably. Where the pipe was laid in, and covered by, clay and loam, more or less impregnated by salt and generally damp, pitting and scaling occurred, as a rule in the bottom of the pipe, the upper half being generally in a uniformly excellent condition.

Pitting is due to a combination of several factors and cannot occur if any one of them is absent. Scaling usually occurs around leaky joints, especially in a salt country. When once started, it only requires moisture to eat itself rapidly into the steel. According to Mr. O'Brien, this is the most dangerous type of corrosion. A perforation arising from pitting alone is generally small in area and closely surrounded by steel of the full original thickness of the plate. It can, therefore, be easily plugged, while a perforation occurring after scaling has started is generally much larger in area and surrounded by thin metal with an irregular surface, which makes repairs much more difficult. The following table gives the number of holes in the pipe caused by external corrosion.

TABLE LXIV. — HOLES DUE TO EXTERNAL CORROSION IN THE COOLGARDIE PIPE

Financial year	No. of holes	Financial year	No. of holes
1904–5	2	1910–11	131
1905–6	27	1911–12	124
1906–7	54	1912–13	774
1907–8	55	1913–14	966
1908–9	91	1914–15	2078
1909–10	177	....	....

A large number of the holes that occurred in the year 1914–15 broke out while the pipes were being scraped, after being unearthed, and would not otherwise have appeared so soon.

Although most of the coating of the covered pipe had been more or less decayed for several years, the steel plates were found to be almost uniformly good, with the exception of the pitting and scaling. According to Mr. O'Brien's report, the total amount of corrosion on the exterior of the worst sections of pipe is small, and does not greatly diminish the strength of the pipe.

Tests of Carrying Capacity. — Tests were made in the different sections of the pipe-line to determine whether the friction opposed to the flow of the water by the inner surface of the pipe was increasing. This friction is due to the roughness of the inner surface of the pipe, which is increased by tuberculation (p. 461). This trouble varies considerably in the different divisions of the pipe-line. According to the inspections made in 1914–15, it appears that along the greater part of the length of the pipe, at least half of the area of the inner surface is free from tubercles, while on the other half they are scattered over the surface at rates varying from 1 to 40 per square foot. Of the remainder of the pipe-line, some sections are entirely free from the nodules caused by tuberculation, while others are almost completely covered by them. These conditions cause wide variations in the friction in the different sections of the pipe.

Experiments have been made of late years to deposit a layer of lime on the inside of the pipe, in order to give the pipe an additional protection against tuberculation. The results thus far obtained are in favor of this lime treatment.

487. *Conclusions about the Corrosion of Iron and Steel.* — While engineers and manufacturers have not yet reached final conclusions as regards the relative resistance to corrosion offered by wrought iron and steel, experience up to the present time seems to prove that wrought iron pipes will outlast those of steel, if both are buried in the ground.

A number of chemists have tried to prove the contrary by *accelerated corrosion tests*, made in their laboratories, in which specimens of wrought iron and steel were subjected to the actions of sulphuric acid. In some of these tests steel resisted the action of the acids better than wrought iron, but it has not yet been proved that these tests are equivalent to exposure to the elements for a period of years, either in the ground or on the surface. On account of its cheapness and greater strength and elasticity, steel is generally used in preference to wrought iron for water pipes.

CHAPTER XXVII

ELECTROLYSIS

488. Chemical decomposition caused by electricity is called electrolysis. Resulting from stray electric currents from trolley railways, it is frequently the cause of serious corrosion of iron and steel pipes. This subject has been very thoroughly discussed by Prof. Albert F. Ganz, M. E., in a paper on "Electrolysis from Stray Electric Currents," * from which the following facts have been abstracted.

In a double trolley railway system, which is rarely used on account of the expense it involves, the return current from a trolley car passes through a separate wire to the power-station, and no electrolysis can occur. In the single trolley system, the rails are used as conductors for the return current. If the rails are properly insulated by being

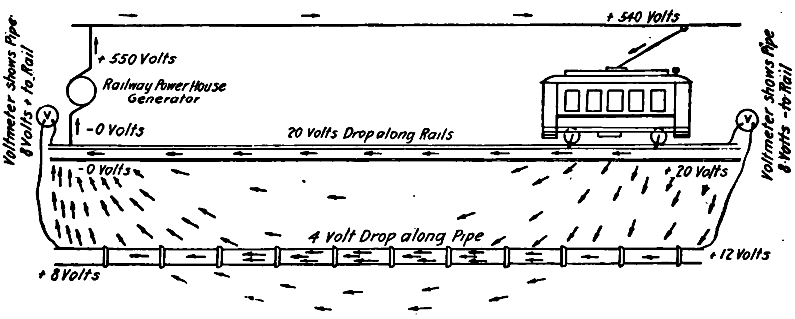


FIG. 263. — Stray Electric Currents from a Trolley Line.

spiked to wooden ties, which are laid on a ballast of broken stone, no trouble occurs; but if the rails are in contact with a soil containing salts, and especially if this soil is moist, part of the return current will pass through the soil to any nearby iron pipe, which leads in the right direction. The stray current will follow the pipe-line until it reaches a point — usually near the power-station — where the rails offer less resistance to the electric current than the pipe. The current then leaves the pipe, passes through the ground to the rails and through them to the power-station. Fig. 263, reproduced from Professor Ganz's paper, illustrates the paths followed by the stray currents. This straying of part of the current to a water-pipe and, later, back again to

* Jour. Amer. Water Works Assoc. for 1912, p. 159.

the rails, does not cause any loss to the railway company, but is, on the contrary, an advantage, as the total conductivity of the return circuit is increased, and the voltage loss in the return current is thus diminished. The railway company gains in this manner power, while a serious damage is done to underground pipes, lead cables, etc.

No electrolysis occurs where the stray current from the rails reaches a pipe; but serious corrosion, depending upon the composition and condition of the soil, takes place where the current leaves the pipe to return to the rails. In some cases, electrolytic corrosion has eaten through iron pipes in three or four years, even when the pressure of the current was only 1½ volts. Electrolysis of wrought iron or steel pipes usually causes *pits* which eventually go through the whole wall of the pipe. In cast iron pipes, corroded by electrolysis, the oxides of iron, mixed with graphite, usually remain in place, so that the outward appearance of the pipe is unchanged. In such cases, however, the material resulting from electrolysis, which usually has the consistency of hard graphite, can be cut with an ordinary knife.

Pure water or dry soil will not conduct electricity, but when water contains salt, even though the percentage is small, or when the soil is moist, electricity will pass easily through either of them by electrolytic conduction.

If the joints of a pipe-line offer a higher resistance to the stray current than the soil, the current will pass around the joints through the soil, causing electrolysis at each joint, where the current leaves the pipe. While the corrosion caused in this manner at the joints is usually much less than where the current leaves the pipe-line, to return to the rails near the power-station, it may still cause serious trouble.

From investigations made thus far, it appears that the corrosion produced by electrolysis is independent of the voltage, except so far as it determines the amount of current flowing, and that the smallest fraction of a volt can produce corrosion from electrolysis under suitable conditions.

The conclusions of Professor Ganz about electrolysis from stray electric currents are as follows:

(1) There is only one complete remedy for electrolysis, namely, the use of a completely insulated return circuit. This may be obtained by double overhead trolley wires, by an insulated out-going and return current in underground conduits, or by separate insulated third and fourth rails for the out-going and return circuit — a system used on the Metropolitan District Railway of London.

(2) Where serious electrolysis is caused by stray currents from

single trolley street railways, the greater part of the trouble is due to defective rail bonding, to ground connections from the negative bus-bar, and to lack of return feeders to bring the current back from the rails to the power-station.

(3) Large stray electric currents from railways can always be reduced to a small fraction of their former value, by removing all ground connections of the negative bus-bar, and installing insulated return feeders proportioned for equal drop from radially disposed points in the track system, located at some distance from the power station. This system, which is in general use in Europe and in a number of American cities, removes the root of the trouble by draining the rails of current, and removing voltage drop from the rails, thus preventing leakage of current through the ground.

Professor Ganz states that if the railway companies would apply as much engineering knowledge and money to their negative circuits as they do in their positive circuits there would be but little trouble from electrolysis.

489. Prevention of Electrolysis. — As already stated, the only sure means of preventing electrolysis is by providing completely insulated return circuits. Various other ways of preventing electrolysis, or at least of reducing its effect, have been tried. Different kinds of paints, dips and insulating covers have been put on pipes for this purpose, but have failed in wet soil to prevent electrolysis. The only kind of insulating covering which appears to afford certain protection is a layer, at least 1 to 2 ins. thick, of a material like coal-tar or asphaltum, of such a grade that it is not brittle and yet hard enough to remain in place. As an additional protection, an insulated coupling should be introduced at each end of the section covered so that, even if the covering should become defective at any point or points, no current could reach the pipe and corrode it. The cost of such a protection would be in most cases prohibitive.

Covering a pipe with cement or concrete has been tried as a preventive of electrolysis, but experience has shown that even when this coating was several inches thick the pipes were corroded as rapidly by electrolysis as when they were simply embedded in the ground.

Current flow on pipe-lines can be practically prevented by using insulating joints for every joint. Such joints may be obtained by filling the ordinary joints of bell-and-spigot cast iron pipes with cement, providing the contact of the spigot with the metal of the bell is prevented. While cement is as good a conductor as ordinary soil, it has, compared with iron, a sufficiently high resistance to practically interrupt the electrical continuity of the pipe-line. Electrolysis may occur even in pipes provided with cement-joints, but in such cases the stray currents reach the mains or service-pipes from other pipes or by other paths.

Creosoted wooden blocks have been used in pipe-joints, instead of cement, to insulate the joints. Fig. 264 shows the Macallen joint which has been used very successfully for insulating small wrought iron or steel pipes. A type of insulating joint applicable to flanged pipes is shown in Fig. 265. In this joint a fiber disc is placed between the surfaces of the flanges, the bolts are insulated with fiber tubing and the bolt heads and nuts are insulated with fiber washers.

490. An electrolysis survey should be made by expert electricians whenever there is good cause to think that underground pipes may be damaged by stray electric currents from electric railways. Such currents coming from telegraph or telephone systems are too weak to cause any trouble by electrolysis. Direct-current electric lighting systems in which the distribution is on the Edison 3-wire plan with the neutral conductor grounded are in American practice provided with such large neutral conductors of copper that practically no stray currents are produced from such systems. This grounding of the neutral conductor in such systems is to serve as a safeguard, and is not for the purpose of using the ground to carry current. Alternating-current lighting systems, where grounded, generally also produce only small stray currents, and the electrolytic effects from these small stray alternating currents are always negligible.

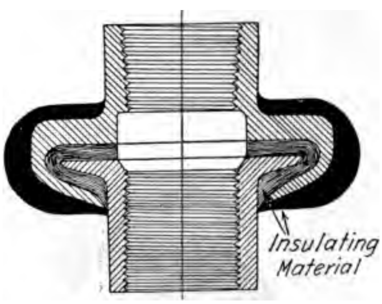


FIG. 264. — Macallen Insulating Pipe-joint for Service-pipes.

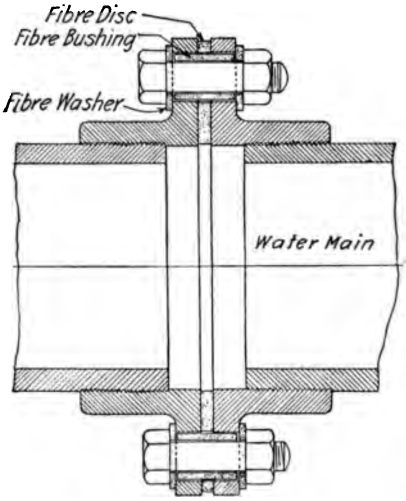


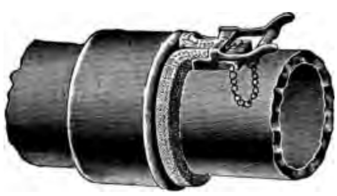
FIG. 265. — Insulating Flange-joint for Water Mains.

CHAPTER XXVIII

TOOLS AND MACHINES FOR PIPE WORK

491. Tools for Pipe-laying. — Some of the tools which are required in laying cast iron pipes are shown in Figs. 266 to 269. In making a hub-and-spigot joint, hemp yarn is first driven into the joint by means of the yarning tool, so as to leave just enough space for the required depth of lead, which varies in ordinary cases from about $1\frac{3}{4}$ ins. for a 4-inch pipe to $2\frac{1}{2}$ ins. for a 48-inch pipe. After the yarn has been driven into the joint, a piece of rope covered with plastic clay is wound around the pipe at the joint, in order to keep the lead in the joint, when it is poured; or what is known as a pipe-jointer can be used. In Fig. 266, *A* shows such a jointer in position

ASBESTOS PIPE JOINTER
SQUARE



A

CLOSED

B

FIG. 266. — Pipe-jointers.

and *B* is another type of jointer, made of a special kind of packing, which is reinforced by a brass band. This jointer can be quickly adjusted against the bell of a pipe, and is held firmly in position by means of a clamp, attached to one end of the jointer.

After the lead has been poured the surplus of lead is cut off with the lead cutting chisel, and the joint is caulked by means of the caulking irons and a hammer weighing about $3\frac{1}{2}$ lbs. (Fig. 267). The thickness of the caulking irons at the point varies from $\frac{1}{8}$ to $\frac{5}{8}$ in.

Cast iron pipes have occasionally to be cut, to obtain short lengths of pipe, or to remove a cracked part of a pipe. In such cases the pipe is placed on wooden blocks and revolved slowly while a slight cut is made around the circumference by means of a diamond-point. This cut is gradually enlarged by the diamond-point, dog-diamond, and pipe-cutting chisels, and after the cut has been made deep enough

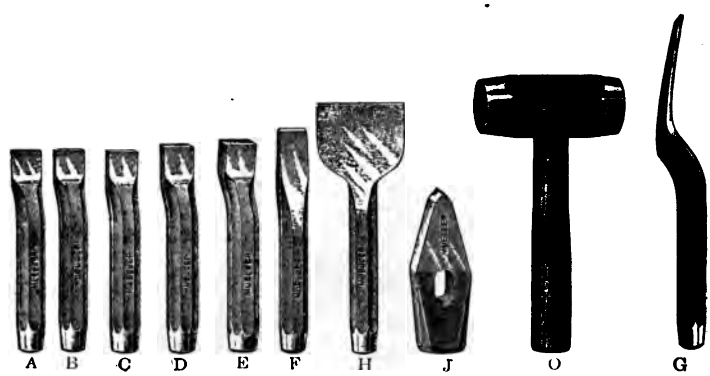
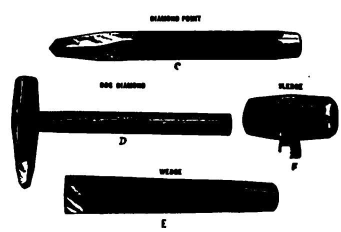


FIG. 267. — Set of Caulking Tools.
A to F, caulking irons; F, cold chisel; G, yarning iron; H, lead cutting chisel; J, sledge; O, caulking hammer.

around the circumference of the pipe, it is not difficult to snap off part of the pipe, either by placing a block beneath the pipe, at the point of the cut, and pressing down at either end; or by driving a

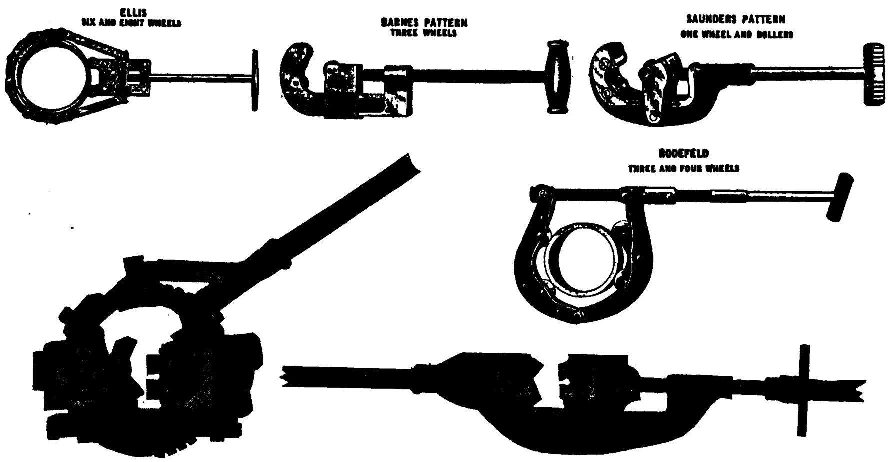


C — diamond-point;
D — dog-diamond;
F — sledge;
E — wedge.

FIG. 267*a*. — Pipe-cutting Tools.

bursting wedge into the cut, causing a crack about the pipe, where previously marked by the diamond point. The tools used for cutting a pipe are shown in Fig. 267*a*.

492. *Pipe-cutters.* — Several machines have been invented for cutting cast iron pipes in a more expeditious and convenient manner than is possible by the simple method described above. Some of these machines are shown in Fig. 268. The Rodfeld, Barnes and Ellis pipe-cutters can be used for cutting cast iron pipes up to 12 inches in diameter. The French pipe-cutter * (Fig. 268*a*), invented by D. W. French, cuts pipes having diameters from 2 to 48 ins. The



BEAVER CUTTERS

FIG. 268. — Pipe-cutters.

machine is made in five sizes, varying from 4 to 48 ins. Each machine can be used for at least three sizes of pipe.

The Saunders and small Beaver pipe-cutters (Fig. 268) are used for cutting wrought iron or steel pipes, 2 inches or less in diameter. Pipes of this kind up to 6 ins. in diameter can be cut by the larger Beaver pipe-cutter.

493. *Pipe-end Reamers.* — Fig. 269 shows a pipe-reamer which is used

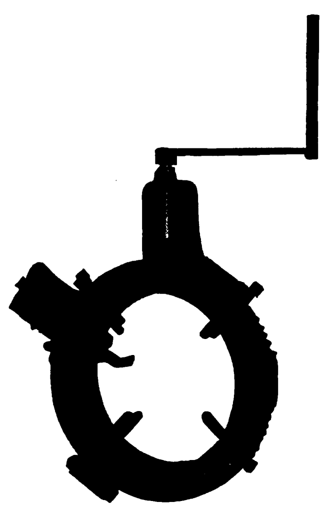


FIG. 268*a*. — French Pipe-cutter.

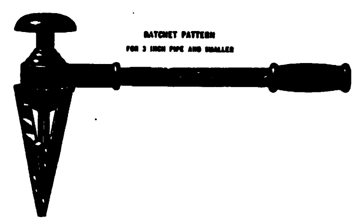


FIG. 269. — Pipe-end Reamer.

for restoring the original inside diameter of a wrought iron or steel pipe after cutting. Without the use of this tool, a certain per cent of the carrying capacity of the pipe is lost, as a result of the obstructions

* Manufactured by the A. P. Smith Manufacturing Co., East Orange, N. J.

resulting from the pipe's being forced inward under pressure of the cutter. The construction of the reamer admits of its use on any pipe from $\frac{3}{8}$ to 3 inches in diameter.

494. Neil caulking machine (Fig. 270 *) is designed for machine caulking on pipes, 24 inches or more in diameter. It works either by steam or compressed air, and can be operated by one man and a boy. The machine works uniformly around the pipe, and makes a circuit of the joint with each tool, before beginning with the next. The power of the blow can be varied from one ounce up to 100 lbs. The work done by this machine is said to be superior to the best hand work and considerably cheaper. On a 24-in. pipe-line more than forty joints can be caulked per day. The machine caulks as well at the bottom as at the top of the pipe.

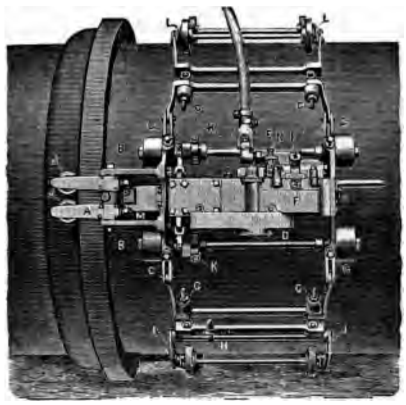


FIG. 270. — Neil Caulking Machine.

495. Melting furnaces of various kinds are used for melting the lead required for filling the pipe-joints. In the old style of furnace (Fig. 271), the fuel used is wood, coke, or charcoal. This kind of furnace is heavy and unwieldy. Fig. 272 shows a furnace in which gasoline is used as fuel. It has three burners, which must be replaced, when they become clogged with the impurities which are in the gasoline.

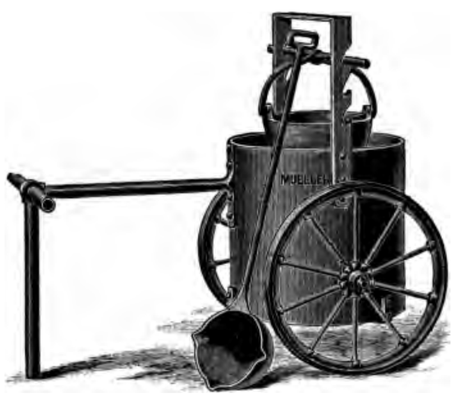


FIG. 271. — Lead Melting Furnace.

FIG. 272. — Gasoline Lead Furnace.

* Manufactured by the A. P. Smith Mfg. Co., East Orange, N. J.

In operating this furnace, the gasoline is placed in the retaining tank at the bottom, and by means of air pressure, produced with a rubber bulb, gasoline is fed through the three burners, vaporized and burning, producing an intense heat.

Other styles of lead melting furnaces, using kerosene, or even heavier oils, are manufactured. Figs. 273 and 273*a* show a portable lead furnace,* using kerosene as fuel. The tank is filled with kerosene oil to the level of the air valve *F*. The burner *C* is heated by igniting a little kerosene in the dish *B*, the chimney *I* being used to concentrate the heat on the burner. The pump *G* is then given a few strokes to force the oil into the burner *C*, where it is converted into gas, which issues from the jet *D*. The gas is ignited, the chimney is removed by the handle attached to it, and the pumping is continued until the gauge shows a pressure of ·20 lbs. per square inch, which suffices for one hour.

FIG. 273. — Portable Kerosene Lead Furnace.

The tank with the burner is placed under the furnace, the lugs *HH* engaging with corresponding lugs on the bottom of the furnace. The heat can be regulated by the air valve *F*. The manufacturers claim that this furnace will melt a pig of lead in 24 minutes, and a second pig in 12 minutes.

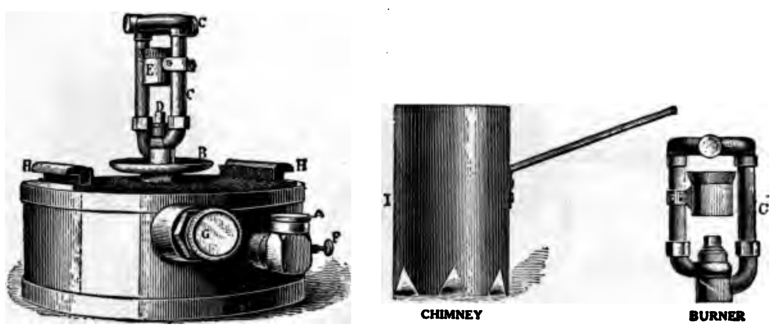


FIG. 273*a*. — Details of Furnace.

Fig. 274 shows a small lead furnace * which can be placed on a water main, so that the molten lead can be poured directly into the joint, instead of by means of a ladle.

* Manufactured by the A. P. Smith Mfg. Co., East Orange, N. J.

Occasionally an old lead joint has to be melted out, in order to make it possible to remove a pipe. The Buckeye heater (Fig. 275 *) can be used advantageously for this purpose. Kerosene or gasoline can be employed as fuel in this furnace.

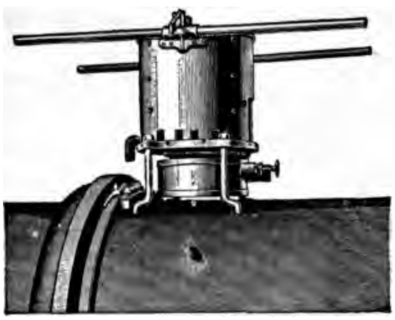


FIG. 274. — Small Portable Lead Furnace.

FIG. 275. — Buckeye Heater.

496. The Smith Tapping Apparatus † for making branch connections, without shutting off the water from the main, is shown in Fig. 275. It can be used for all sizes of pipe connections that occur

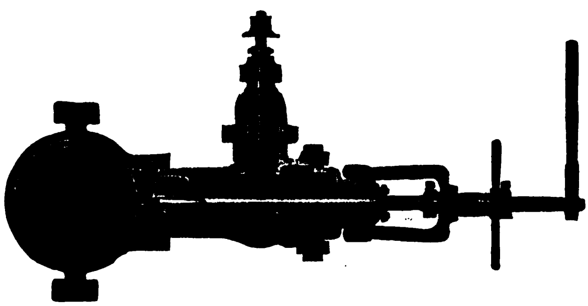


FIG. 276. — Smith Tapping Machine for Branch-connections.

generally on waterworks, such as 48 by 36 ins., 36 by 36 ins., 36 by 30 ins., 30 by 30 ins., etc.

In using this apparatus the first step is to remove all rust and dirt from the pipe where the connection is to be made. A split sleeve (Fig. 276) is then bolted to the main at this place, and its

* Manufactured by Walter J. McLeod, Cincinnati, Ohio.

† Manufactured by the A. P. Smith Manufacturing Co., East Orange, N. J.

joints are poured with lead and caulked. The spigot end of the valve which is to be placed on the branch-connection is inserted in the hub-end of the split sleeve and locked by a slight turn to the left. The joint between the split sleeve and the valve is then poured and caulked. An extension piece corresponding to the size of the valve is connected to the machine, and then to the valve, both joints being made with rubber gaskets.

The valve is opened wide and the shaft of the machine is pushed forward until the drill is against the pipe. The set-collar on

FIG. 277. — Split Sleeve and Gate-valve for Branch-connections.

the shaft is secured by means of a set-screw. The shaft is now rotated slowly to the right by means of the ratchet-wrench until the cutter begins to cut the pipe, when the lever handle on the ratchet-wrench is used.

After the cut has been made, the set collar is released, and the shaft is withdrawn, bringing the drill and cutter back of the valve which is now closed. The extension pipe is disconnected from the valve and the connection is completed.

497. The Smith valve inserting machine* is used for inserting gate-valves in pipes, 8 to 36 ins. in diameter, without shutting off the water. The manner in which this operation is performed is as follows: First, a double-headed pipe-cutting machine is placed on the main, where the valve is to be inserted, and the pipe is cut about half way through

* Manufactured by the A. P. Smith Mfg. Co., East Orange, N. J.

at two points. A special split sleeve is then bolted around the pipe, with one end within $\frac{1}{4}$ in. of the cutting tool, and the *inserting machine* (Fig. 278) is bolted around the pipe and sleeve, enclosing the cutting mechanism completely. The joints between the pipe and the machine casing are leaded and caulked, and the pinion shaft is rotated until the pipe has been cut through.

The cutting machine with the section of pipe it has cut out is drawn up into the dome of the machine, by means of a central rod; the horizontal slide-valve is closed; the top of the dome is removed, and the cutting machine and pipe-section are taken out. The gate-valve that is to be inserted is now placed in the dome of the machine, the cover is attached; the slide-valve is opened and the gate is lowered into the place between the ends of the pipe. The split sleeve is forced by jacks into the large hub of the gate-valves, and the motion is continued until the small hub of the gate-valve has been forced over the pipe. Both hubs are provided with wedge-shaped lugs, which center the pipe and sleeve ends and hold in place the rubber gaskets which make the temporary joints between the pipe ends and the gate-valve. The upper casing is removed and the joints at both ends of the valve and the sleeve joint are leaded and caulked.

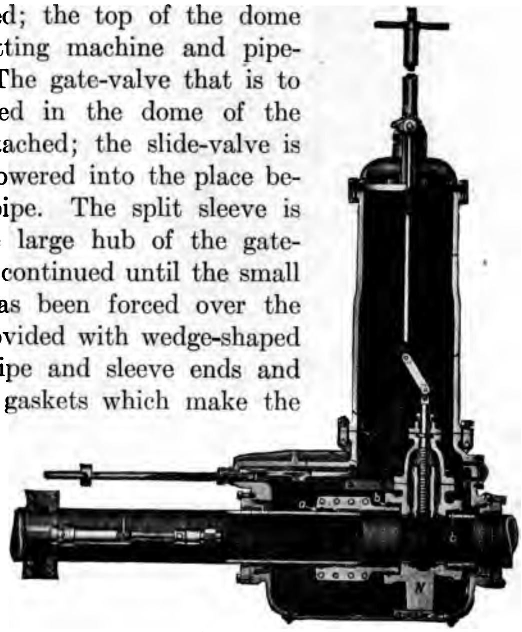


FIG. 278. — Smith Valve Inserting Machine.

498. Pipe Derricks. — Pipes of small diameters can be easily lowered into position in the ditch by means of rope-slings, but this method of handling becomes almost impossible with the larger sizes of mains. Fig. 279 shows the method of lowering single lengths of pipe into the ditch by means of a derrick, tongs and a chain hoist. This method naturally requires much less labor. If joints can be made above ground, the workmanship will be better, and to facilitate this method of laying, two to five sets of derricks, pipe-tongs and chain-falls can be used. An apparatus of this kind presents great labor and money saving possibilities. With five sets of these derricks and pipe-tongs, ten lengths of twelve-inch main, one hundred and twenty feet in all, can be caulked above the ground and lowered as a unit.

Fig. 279. — Pipe Derrick.

499. Tapping machines are used for inserting corporation cocks in water mains, while they are under pressure. The machine drills a hole in the pipe shell, cuts a thread in it, and screws the corporation-cock into position. The manner in which this work is done is explained in detail in the descriptions of some of the principal tapping machines given below.

500. *The Mueller tapping machine* (Fig. 280 *) makes taps and inserts corporation-cocks, $\frac{3}{8}$ to 1 in. in diameter, in mains under pressure, and taps up to 2 ins. in diameter in pipes that are dry or open. It is a strong machine with long bearings, made for constant and heavy usage. The body part *F* is fastened to the pipe, which is to be tapped by an adjustable chain *G*. A boring-bar *B*, provided with a ratchet-handle *L* for turning the bar, receives the drill and the tap *J*. The tool *J* is forced down or fed by turning the hand-wheel *K*, which, by means of the yoke *M*, puts the proper pressure on the collar *N*. The body of the machine is provided with a gate *O*, which serves for closing the upper part of the machine from the water, after the boring-bar and tool have been drawn up from the pipe, to replace the tool by the corporation-cock. After the gate *O* is closed, the water in the

* Manufactured by the H. Mueller Mfg. Co., Decatur, Ill.

upper chamber of the machine is discharged though the by-pass *R*, and the pressure in the main keeps the gate *O* tightly on its seat. This by-pass *R*, also, enables the operator to see if the corporation-cock is properly screwed in, before removing the screw-plug. The machine is operated in the following manner. The cap *A* is removed.

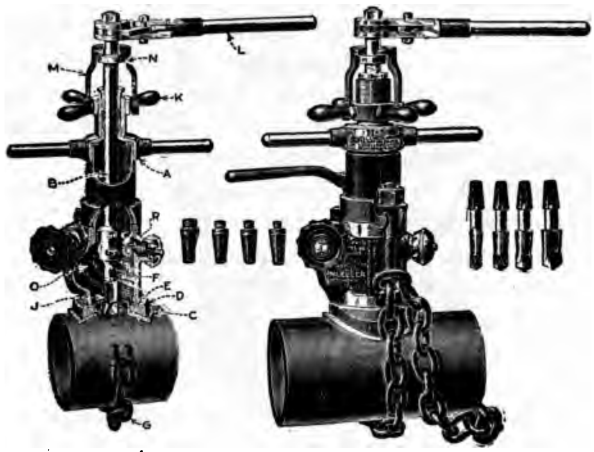


FIG. 280. — Mueller's Tapping Machine.

This takes out also the boring-bar *B*. The rubber gasket *C* is then adjusted in the curve of the saddle *D* and the rubber gasket *E* is placed in the saddle. The body of the machine *F* is then placed upon this gasket. If the tap is to be made on the side of the pipe, instead of in a vertical line, the body *F* must be set so that the word *Top* is up. The machine is next fastened to the main by means of the chain *G*, which is put around the pipe. One link of the chain is hooked over the bolt *I*, which is just opposite to an eye-bolt, to which the other end of the chain is permanently fastened. By means of the nuts on these bolts, the machine is drawn tightly to the main, by screwing down the nuts alternately. The machine must be fastened very firmly, as a slight movement in any direction is liable to cause serious trouble, and to prevent the accurate centering of the corporation-cock. The tool *J*, which serves both for drilling and tapping, is fastened in the holder *B*, by means of a set-screw. In operating the machine, the hand-wheel *K* is turned to the right, as far as it will go, and the boring-bar is put into the body of the machine. Cap *A* is screwed down until the tool touches the pipe; the ratchet-handle is placed in position and turned, the tool being fed by means of the hand-wheel *K*. When the tap *takes hold*, further feeding with the hand-wheel is unnecessary.

After the hole has been properly drilled and tapped, the dog on the ratchet-wheel is reversed, the feed-yoke *M* is thrown off the collar *N*, and the tool is backed out of the pipe, care being taken, especially with high pressures, not to allow the tool to be forced out too suddenly, as this might strip the thread and strike against the top of the machine. The boring-bar is pulled up, as far as it will go, and the gate *O* is closed by turning the knob *P* to the left or up. By means of the by-pass *R*, the pressure is released from the upper chamber of the machine. Cap *A* is then unscrewed, the boring-bar is removed and the tool is replaced by the screw-plug *Q*, upon which the corporation-cock has been screwed. Cap *A* is now screwed down again, the by-pass and gate are opened, and the boring-bar, with the corporation-cock attached, is pushed down, while the ratchet-handle turns it to the right until the threads take hold. As soon as the cock has entered the tap-hole, no more pushing is required. After the cock has been screwed far enough into the main — which can be known by the squeaking of the threads — the dog on the ratchet-handle is reversed, and a sharp blow on the ratchet-handle releases the plug *Q*, which is then unscrewed from the corporation-cock. Before unscrewing the plug *Q*, however, the by-pass *R* should be opened to ascertain whether the corporation-cock is properly set. After the plug *Q* has been unscrewed and before removing the machine, the by-pass *R* should be opened. If there appears to be a considerable flow of water, it shows that in removing the screw-plug *Q*, the corporation-cock was, also, unscrewed. If this proves to be the case, the operation of inserting the corporation-cock must be repeated. When the test shows that no water is flowing from the main through the tap-hole, the machine may be removed.

501. *A. P. Smith tapping machine* (Fig. 281*) is made in three sizes: No. 1 is used for $\frac{1}{2}$-, $\frac{5}{8}$-, and $\frac{3}{4}$-in. corporation-cocks; No. 2 for 1-, $1\frac{1}{4}$-, and $1\frac{1}{2}$-in. insertions, and No. 3 for 2-in. cocks. The tapping and making of the connections is done without interrupting the flow in the main. The machine is operated as follows: The corporation-cock *H* is screwed into the carrier *I*, which is placed in the slide *J*. The tap *F* is put into place, and the machine is then attached to the main, as shown in Fig. 281. During the operation of drilling and tapping the main, the feed should be continued until the face of the ratchet-wrench *D* is brought in contact with the gland *E*, which determines the depth to which the corporation-cock is to be inserted in the pipe. The drill-shaft is then withdrawn, as far as possible, and the wing-nut *G* is turned half a revolution, which brings the corpo-

* Manufactured by A. P. Smith Mfg. Co., East Orange, N. J.

ration-cock into position for insertion. The drill is then allowed to drop into the carrier, and the cock is screwed into the main.

502. *Payne's "New Eclipse" tapping machine* (Fig. 282*) taps a pipe on the top, the side, or at any desired angle. The machine is

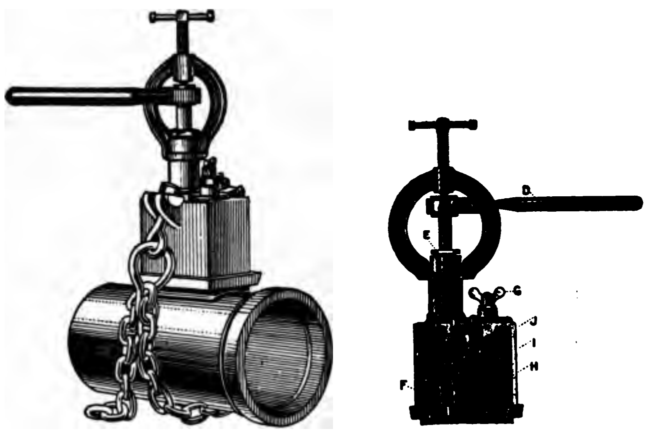


Fig. 281. — Smith Tapping Machine.

securely fastened to the pipe by means of a chain with end-bolts, a thick rubber gasket being placed between the pipe and the saddle of the machine to obtain water-tightness. The desired size of taps is put in this machine, and the corresponding corporation-cock placed, closed, on the mandrel. A thin rubber gasket is put into the saddle and the machine is screwed tight on the saddle by means of the iron handle on the machine. The revolving-head is next turned back to the right until it strikes the stop.

Fig. 282. — Payne's Tapping Machine.

The drill is pushed down until it rests on the pipe; the ratchet, and the yoke over it, are put on, and the tap-hole is drilled. When this hole has been tapped, the drill is drawn up in the machine, as far as possible, a lengthening piece is screwed onto the mandrel, and the head is revolved to the left, until it strikes the top. The cock is pushed into its place, and screwed in by means of the mandrel just far enough to be secure, while the machine is being taken off.

* Manufactured by the Hays Mfg. Co., Erie, Penn.

The bolts attached to the chain are loosened, and the machine is removed, leaving the mandrel in the cock, which is now screwed tight with a wrench. The mandrel is finally removed from the cock, which may then be connected to the service-pipe.

The machine is made of the best bronze metal and the saddle of malleable iron. It is simple in construction, having neither valve nor pet-cock, to get out of order. The pressure of the feed-screw pulls directly on the chain, thereby relieving the machine of all undue strain. Two sizes are manufactured, *viz.*:

Number	Weight	Size of taps, inches	Saddles
$1\frac{1}{2}$	34	$\frac{5}{8}$, $\frac{3}{4}$ 1 and $1\frac{1}{2}$	4
$2\frac{1}{2}$	76	1, $1\frac{1}{4}$, $1\frac{1}{2}$ and 2	4

$\frac{5}{8}$-in. corporation-cocks with iron pipe thread on main end are inserted with $\frac{3}{4}$-in. tap. The machines are regularly equipped with female mandrels, but can be supplied with male mandrels if required.

503. Wireless Pipe Locator. — The Modern Iron Works of Quincy, Ill., manufacture "The Wireless Pipe Locator" shown in Fig. 283.

Fig. 283. — Wireless Pipe Locator.

This consists of a vibrator, an induction coil, batteries, and a detecting coil to which is attached a telephone receiver. The terminals of the primary coil are attached through the vibrator to the battery terminals. The vibrator interrupts or changes the strength of current through the primary coil, and the voltage is increased by the secondary coil, the terminals of which are fastened to two places on the pipe or main to be located. They are connected, for instance, to a hydrant and a house-service. The changes of current set up an electric flow in the pipe or main, and the variation in intensity of the electric field about the pipe or main causes a variable flow by induction in the detector coil, which causes the telephone receiver to *sing*. The nearer the coil is brought to the pipe or main, the louder the tone, if the detector coil is held horizontally, until it is directly over the pipe or main, when no sound is produced. At this point, if the detector coil is turned into a vertical position, the loudest tone is given.

Fig. 284 shows the manner in which this useful instrument is generally used. No skill or knowledge is necessary to get good results, and the saving in expense and time, when it is desired to locate a main or pipe, usually makes the use of this instrument desirable.

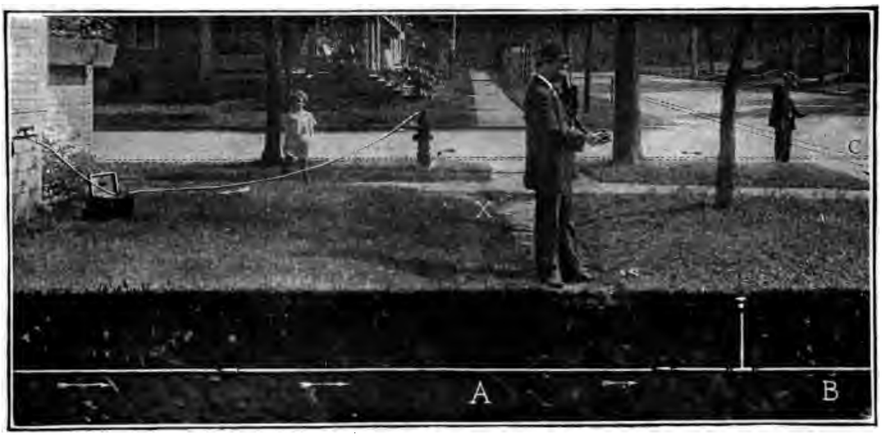


FIG. 284. — Manner of Using Wireless Pipe Locator.

A and *B* — service-pipe, iron and lead; also, the stop-box or cut-off, covered by the grass.

C — the mains in the street.

X — the field. The arrows show the direction of the current.

503a. The Grove Electric Indicator, invented by E. H. Grove, and manufactured by the A. P. Smith Mfg. Co., East Orange, N. J., is another device for locating underground service-pipes and mains without excavation. This indicator, which was perfected in 1912, has been used very successfully in Washington, D. C., and in other cities.

CHAPTER XXIX

THE DETECTION AND PREVENTION OF WASTE OF WATER

504. Waste of Water. — A large percentage of the water supplied by public water works is wasted by leaky mains, service-pipes, and plumbing fixtures, and, also, in many places by allowing water to flow continuously through faucets in winter, to prevent the water from freezing in the service-pipes, and in summer to obtain cool water for drinking. In American cities the loss of water has been roughly estimated to be about 30 to 50 per cent of the consumption.

The introduction of meters into private, as well as into public, buildings, checks willful waste. There still exists a strong prejudice in America against measuring the water used for private consumption, and the argument is often advanced that this restricts the free use of water and is, therefore, unsanitary. The answer to this objection is that the quantity of water allowed at the minimum rate should be made large enough to permit all legitimate uses of water, and that any excess beyond this should be paid for additionally. The smaller cities in the United States are beginning to place meters on all service-pipes, and, in due course of time, water meters will, doubtless, be placed not only in all houses, but in every flat or apartment, just as gas meters are now installed.

Even where the total supply of water furnished, and, also, the consumptions are accurately measured, there is usually a difference of about 20 to 40 per cent between these two quantities. This represents the underground leakage in mains, service-pipes, gate-valves, stuffing-boxes, etc.

The losses of water by leakage in the pipe-system, or in the plumbing of houses, can be discovered by systematic investigation, by dividing a city or town into districts of convenient size, and measuring the quantity of water consumed in each, by means of water meters or other suitable instruments. If the consumption in any district, especially between midnight and 4 P.M., is found to be abnormally large, the district must be subdivided into sections, the consumption in each being investigated in a similar manner as that of the whole district, until the particular block or house is found, where the waste of water occurs. One of the first cities to make such an investigation was Liverpool, England.

505. Prevention of Waste of Water. — In 1870, the city of Liverpool was suffering from a scarcity of water, which necessitated shutting off the supply for about 14 hours out of 24, the citizens being thus obliged to depend upon an intermittent service. After remedying all defective plumbing by a house to house inspection, the engineer in charge of the water works, George F. Deacon, M. Inst. C. E., determined to look for underground leakage and to suppress it, if found. For this purpose, he divided the city into a number of districts and measured the flow into and out of each, by means of a special meter, which he invented for this purpose (p. 541). Where the consumption in a district seemed abnormally high, the district was subdivided, and the consumption in each of the smaller districts was determined in a similar manner. In this way, underground leaks were traced and stopped, and individual houses found in which too much water was consumed. By this method, the water supply was conserved to such an extent that it was restored in 1875 to a constant service, after an intermission of five years, and the daily consumption per capita was reduced from 35.58 gals. in 1873 to 14.26 gals. in 1877.

Since its first introduction in Liverpool, Deacon meters have been largely used in Europe for discovering water waste in public supplies, and they have, also, been used to some extent in Boston, Mass. Simpler and less expensive instruments have, however, been devised for accomplishing the same purpose, and have been largely used within the past decade in the United States.

506. *Prevention of Waste of Water in Washington, D. C.*—The city of Washington, D. C., has been engaged for some years in discovering and suppressing leakage in water mains, etc., by means of modern methods. The following brief summary of what was discovered in the year ending June 30, 1911, gives some idea of the different causes of leakage.

TABLE LXV. — DISCOVERY OF LEAKAGE IN THE CITY OF WASHINGTON, D. C., FOR THE YEAR ENDING JUNE 30, 1911

Description	No.	Gallons wasted per day
Abandoned service and taps leaking	11	305,000
Iron service-pipes broken	204	2,438,000
Lead service-pipes broken	87	1,202,000
Wiped joints broken	74	710,000
Couplings on service-pipes leaking	18	119,000
Curb-cocks leaking	30	85,000
Tops blown out	3	50,000
Joint on mains leaking	92	1,034,000
Mains broken	2	332,000
Valves leaking	11	89,000
		6,364,000

507. *Prevention of Waste of Water in New York.** — In 1911 the city of New York was obliged to adopt strenuous measures to avoid a water famine. During the first half of 1910 the rainfall had filled all the storage reservoirs of the city, but from June, 1910, to the latter part of August, 1911, the precipitation was less than had been recorded for the same period of time during the preceding forty-three years. The large quantity of water stored in the reservoirs made it possible to maintain an ample supply during 1910, in spite of the drought, but by the beginning of 1911, the quantity of water in the storage reservoir had been so much reduced that radical measures became necessary to curtail the consumption of water, and surveys were made for increasing the supply temporarily by pumping water from Ten Mile River, an affluent of the Housatonic River. The measures adopted for reducing the consumption, which proved to be very effective, were as follows:

1. Public attention was called to the shortage of water through the press.

2. Printed notices were sent to the owners of all houses, ordering them to repair all leaky plumbing fixtures, and notifying that they would be fined if this matter was not attended to. The use of hose inside or outside of the house was prohibited.

3. A house-to-house inspection was made by a force of about 150 special inspectors.

4. The city was divided into a number of districts and a corps of engineers determined the consumption in each of these districts, both before and after the house-to-house inspection. This work was done principally by means of portable pitometers (p. 489).

* Paper by I. M. de Varona on "Work done for the Prevention of Water Waste in the City of N. Y. and Results accomplished Thereby," in Proc. Am. Water Works Assoc. for 1913.

CHAPTER XXX

PITOT TUBE GAUGINGS

508. Pitot tubes, named after M. Pitot, the French hydraulician who invented this device about the year 1730,* are used for measuring the velocity of water in a stream or pipe, by observing the corresponding head. In its simplest form, this instrument consists of a small tube of glass, brass, etc., having its lower end bent at right angles to the axis of the tube. If such a tube is placed in the water with its bent end turned up-stream, and parallel with the current, the velocity head producing the flow will be transmitted through the tube and will balance a column of water in the tube having a height equal approximately to that given by the well-known formula:

$$h = \frac{V^2}{2g}. \qquad (102)$$

To compare this head with the static head of the water, a second Pitot tube, having its bent end turned down-stream, is used. The current of water will, however, exert a slight suction that will lower the water column in this tube a trifle below the normal surface of the water. For this reason we must introduce a coefficient c, based upon experiments, in using formula (101), in connection with Pitot tube observations and the formula becomes

$$h = c\frac{V^2}{2g}. \qquad (103)$$

By means of Pitot tubes the velocity of a stream of water can be observed at different points in its cross-section and the average velocity may thus be found.

509. Recording Pitometer (Fig. 285). — Within the past twenty years, much improvement has been made in the practical application of the Pitot tube principle to the measurement of the flow of water. John A. and Edward S. Cole have invented a simple portable arrangement of Pitot tube, called a rod-meter, a glass *U-tube*, connected to the rod-meter by suitable hose, and a mechanical or photographic recorder, for making a record of deflections in the tubes, to which they have given the name Pitometer.† These instruments have been used very success-

* "Histoire de l'Académie des Sciences," 1732, p. 376.

† Manufactured by the Pitometer Company, New York.

fully in a number of cities in determining the quantity of water flowing through the pipe-system at different points, in order to ascertain the consumption, leakage, waste of water, slips of pumps, etc.

The rod-meter consists of two small brass tubes, $\frac{1}{4}$ in. in outer diameter, which are enclosed in a brass casing having an oval cross-section,

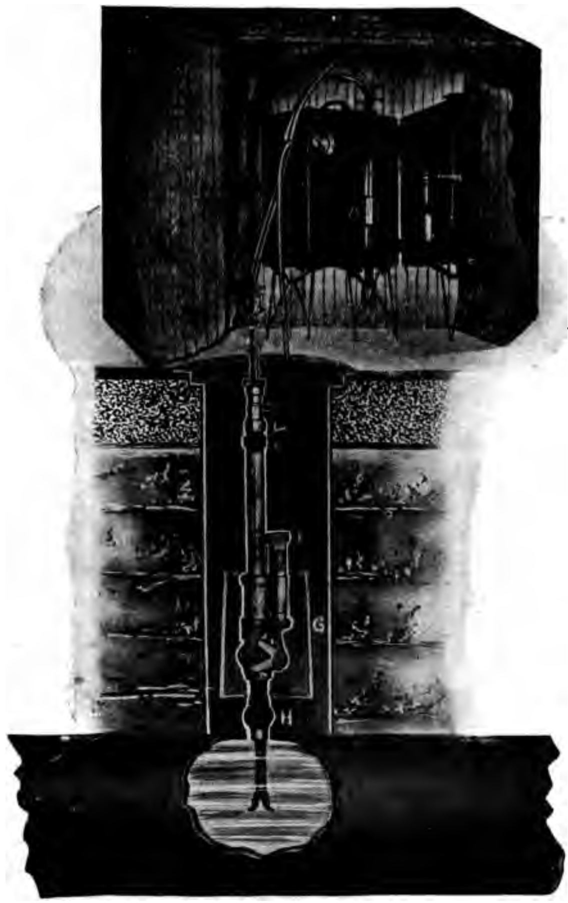


FIG. 285. — Recording Pitometer.

the longer diameter of which is a little less than an inch, to make it possible to push the rod-meter into a water main through a suitable stuffing-box, which is screwed to a standard 1-in. corporation-cock. Each tube has at its lower end a curved phosphor-bronze orifice, $\frac{1}{8}$ in. in diameter, and at its upper end a metal piece called a finger, which is securely clamped to the tube. The fingers serve for turning the Pitot tubes into the desired direction. A loose metal sleeve having two notches for each finger slips over the two fingers and locks them either in the position *open* or *closed*, as the case may be.

When the rod-meter is to be pushed through a corporation-cock into a main, the Pitot tubes must be turned so that their orifices point inward, in order to be able to pass them through the corporation-cock. When the orifices are in the main, they must be turned so that one will point up-stream and the other down-stream, both being in the same plane.

The rod-meter is screwed to the corporation-cock by means of a stuffing-box, a suitable washer being placed in the joint, to insure water-tightness, and, when in position for observation, the longer diameter of the rod-meter must be parallel with the center line of the main. The stuffing-box on the corporation-cock makes it possible to push the rod-meter up or down, without letting any water escape. At the upper end of the rod-meter, each Pitot tube has a small stuffing-box, which prevents the water from passing out along these tubes. The rod-meters are usually about 3 to 8 ft. long, but can be made of any desired length.

The Pitot tubes are connected by heavy cloth insertion rubber tubing with a glass manometer, or *U-tube*, and blow-off cocks are provided for removing air from the instrument. The rubber tubing is fastened to the pitometer tubes and to the U-tube by means of small screw clamps. Two small pinch cocks are placed on each meter tube, and serve as stop-cocks.

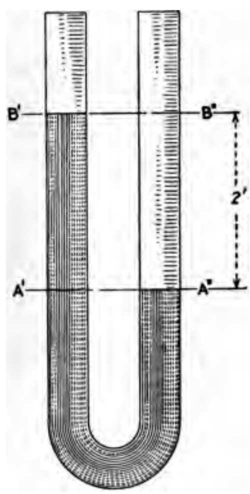


FIG. 286. — U-tube for Pitometer.

The U-tubes are made of different lengths, 2 ft. being the usual size. They are half filled with a mixture insoluble in water, consisting of redistilled carbon tetrachloride and benzene, mixed so as to have a specific gravity of 1.25 or 1.50. A red coloring matter is added to this mixture. The upper half of the U-tube is filled with water. When the rod-meter is inserted in a current of water, the down-stream orifice will have the static pressure on it, whereas the up-stream orifice will be subjected to the static head plus the velocity head. This difference in pressure causes the red liquid in the U-tube to rise in one leg and to sink in the other (Fig. 286). The difference in the levels of the red liquid in the two legs of the U-tube is called the deflection. The two liquid columns are evidently balanced below the line $A'A''$ and, also, above the line $B'B''$. To make columns $A'B'$ and $A''B''$ balance, we would have to add $\frac{1}{4}$ or $\frac{1}{2}$

to the height of the water column $A''B''$, according to whether the specific gravity of the red liquid in the U-tube is 1.25 or 1.50, as these columns are balanced by the velocity head in the water stream, which is evidently equal to $\frac{1}{4}$ or $\frac{1}{2}$ the deflection, respectively, for a specific gravity of 1.25 or 1.50 of the red liquid.

If the pitometer observations are to be continued only for a short time, the deflections of the red liquid in the U-tube may be measured on that tube with a two-foot rule and recorded in a field book. For a longer period of observation — say of 5 to 10 hours — a graphical record of

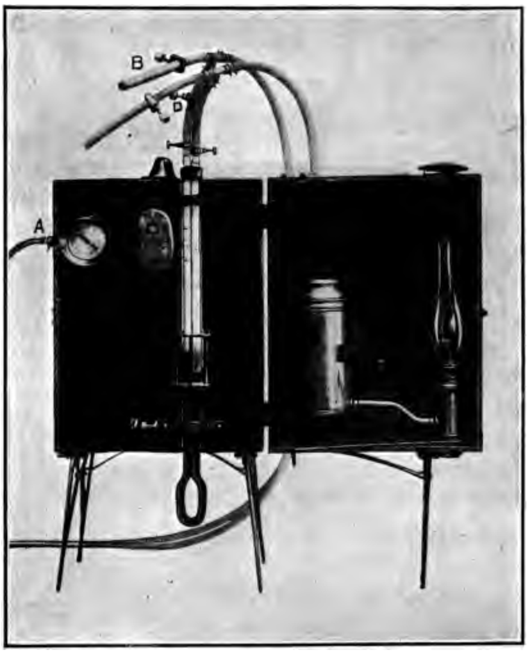


Fig. 287. — Photo-recorder.

the variations in the deflections of the red liquids in the U-tube is obtained by a photographical or mechanical recorder.

510. The Photo-recorder (Fig. 287) is a device for recording photographically the fluctuations of the deflections in the manometer (U-tube) on a chart from which the total flow and its continuous variations may be computed. It consists of a portable light-tight box, containing a drum, carrying sensitized photographic paper,* which is revolved by clock-work in front of a narrow vertical slot between four brass plates, called the shield, two being on each side of the slot, the upper two, as

* Velox paper is generally used.

also the lower two, being attached to each other by a cross-plate. By means of clamps having slow-motion screws, the manometer tube can be attached to the recorder box with its ***high***, or down-stream, leg in front of the slot. An oil, electric, or other kind of lamp is placed on a shelf in the cover of the recorder box, which opens on hinges and is held open in a certain position by means of a hook. The light coming from the lamp is made by proper adjustments to shine through the high leg of the manometer tube and through the slot to the photographic paper. As the manometer tube filled with the red liquid and water forms an elongated lens, it is held by the clamps at a distance from the slot, corresponding to its length of focus. The slow motion screws of the clamps serve to adjust the tube laterally, to make the light from the lamp pass through the slot. As the drum revolves, the red liquid in the tube intercepts the light from the lower portion of the photographic paper, while the upper portion is exposed to the full light passing through the water. Consequently, when the paper is developed, the upper part of the record becomes dark, while the lower part, which was not exposed to light, remains light. The line separating the white from the dark part of the record forms a curve, the ordinates of which correspond to the deflections in the manometer tube. The photographic paper is automatically ruled by the light by means of small notches sawed in the two plates on one side of the slot, on lines parallel with the rays of the light, and spaced to correct for refraction and angularity of the light, so as to represent correctly the deflections in inches. The notches permit a longer exposure of the paper, where they occur, and thus rule lines on the record.

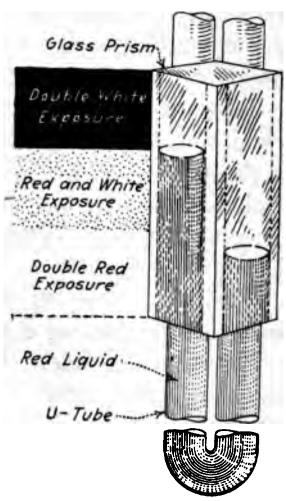


FIG. 287*a*. — Effect of Prism in Photo-recorder.

Changes of temperature cause the liquid in the manometer to expand or contract, and thus to have an effect on the position of the zero or *no-flow* line of the record, which may introduce serious errors in cases of very low deflections. This source of error may be eliminated by photographing simultaneously both legs of the manometer, or by making corrections for the change in specific gravity of the liquid used, due to change in temperature. Both legs of the manometer may be photographed simultaneously by means of a double glass prism (Fig. 287*a*), fastened in front of the manometer, which is designed to refract the light through both legs of the manometer at such angles that the rays will cross in the plane of the slot-plates. The lower two slot-plates can be moved laterally by means of a screw to make the lower part of the slot coincide with the line on which the rays of light from the prism cross.

When the recorder is adjusted so that the light from the lamp shines through the prism, the two legs of the manometer, and the slot, three intensities of exposure are produced, which cause the following results when the photographic paper is developed. As white light has a greater effect on sensitized paper than red light, the upper part of the record on which light shone through clear water becomes black; the middle portion, corresponding to the deflection, which received white light from one leg of the manometer and red light from the other, becomes medium dark, while the lower portion, which received red light from both legs of the manometer, remains white. The width of the medium dark color on the record gives the full deflection of the manometer, without regard to the location of the zero line.

A record like that just described can evidently be obtained only when the deflection is sufficiently small to remain within the range of the prism. For greater deflections only one leg of the manometer is photographed, and the deflection at any point is determined by taking twice the distance between the zero line and the line marked by the difference of shades.

When only one leg of the manometer is to be photographed, the prism can be dispensed with, and the two parts of the slot should then be brought into line. As already stated, the record thus obtained is liable to errors, caused by the expansion or contraction of the liquid in the manometer, due to changes of temperature. For accurate work corrections must be made for these changes. For this purpose a metallic thermometer is attached to the recorder and actuates an arm which moves up and down in front of the slot, producing a white *temperature line* on the record.

511. The Traverse. — The pitometer gives only the velocity at the particular point in the pipe where the orifice of the rod-meter is held. The velocity of the water in any transverse section of a pipe varies greatly, being, under normal conditions, least at the wetted perimeter of the pipe, on account of the retarding effect of friction, and greatest at the center. In order to obtain the average velocity at any given transverse section of a pipe, observations must be taken with the pitometer at a number of points. This is called taking a traverse of the pipe.

The first operation is to determine the exact diameter of the pipe, which is accomplished by shoving the rod-meter downward through the corporation-cock, with the orifices closed, until the orifices touch the bottom of the pipe, and then pulling it upward until the manometer shows no deflection. The extent of the motion can be easily measured, but, in order to get the true diameter of the pipe, the amount of projection of the corporation-cock into the pipe must be added. The exact diameter of the pipe may, also, be found by means of a special caliper-rod which can be pushed through the stuffing-box and corporation-cock into the pipe. The end of this rod is formed by means of two right-angle bends into a hook which can encompass the projection of the corporation-cock without touching it. Knowing the diameter of the pipe, its area is divided into four or five concentric annular rings, having areas that are either equal or approximately so, which are lettered *A*, *B*, *C*, etc. (Fig. 288). Pitometer readings are then taken at the center of the pipe and at the bottom and top of each of the rings, and the results are plotted as shown in Fig. 288.

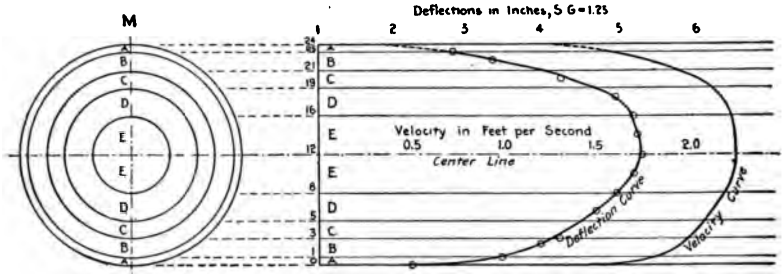


FIG. 288. — Plot of Pitometer Gauging.

The diameters of the annular rings are usually selected with a view to convenience in plotting. By multiplying the area of each ring by the average velocity of the water in this ring and adding the results, we obtain the total discharge of the pipe. This quantity divided by the area of the pipes gives the mean velocity. For velocities that are greater than 1 ft. per second, the ratios of the average velocity to the center velocity is found, for any particular case, to be a constant, which, once determined, enables us subsequently to gauge the flow in the pipe by merely observing the velocity at the center of the pipe.

When there are no disturbing influences, it is sufficient to make one traverse of a pipe on a vertical diameter. Near bends, specials, gate-valves, etc., the traverse curves become much distorted. In such cases two traverses should be taken on diameters at right angles to each other, and the average of the two observations should be used in determining the mean velocity. Even in cases of marked disturbance from normal flow, the ratio of mean to center or to maximum velocity is found to remain practically constant.

Thus far we have assumed the discharge of a pipe to remain constant while a traverse is being made. Considerable variations may, however, occur while the pitometer observations are being made, on account of changes in the rate of consumption, etc. To take these into account, the velocity at the center of the pipe must be frequently taken during the pitometer observations. If variations are found to occur, only those observations corresponding to the same center velocity must be plotted in the same curve. Thus, a number of different traverses may be obtained during the same series of observations, on account of variations in the discharge of the pipe. When observations are to be made on two diameters at right angles to each other, an auxiliary rod-meter may be kept with its orifices at or near the center of one diameter, while a traverse is being taken on the other, the variations in the flow at the center of the pipe being frequently noted.

512. *The office work* in connection with pitometer measurements consists in computing from the field notes the *pipe-coefficient*, which is the ratio of the mean velocity in the pipe to the center velocity, and in developing and interpreting the photographic record. The field notes are plotted on ordinary cross-section paper and give the deflection curve from which the corresponding velocity curve (Fig. 238) can be obtained, by converting the deflections noted in the field into velocity heads by the following formulas:

Let V = velocity in feet per second;
h = velocity head in feet;
g = acceleration due to gravity = 32.16 ft. per second;
c = coefficient based upon experiments = 0.84;
d = deflection in U-tube, measured in inches = $\frac{d}{12}$ ft.;
s = specific gravity of liquid in U-tube.

Taking all dimensions in feet and remembering that the deflection is measured in a liquid having a specific gravity of s, and that the deflection is partly balanced by a column of water of equal height as the deflection in the other leg of the U-tube, formula $V = c\sqrt{2gh}$ derived from formula (102) becomes

$$V = c\sqrt{2g\frac{d}{12}(s-1)} = 2.315\,c\sqrt{d(s-1)}. \qquad (104)$$

Taking $c = 0.84$, the value usually assumed, and $s = 1.25$ and $s = 1.50$, the two specific gravities generally used, equation (104) becomes reduced to:

$$s = 1.25, \qquad v = 0.972\sqrt{d}. \qquad (105)$$
$$s = 1.50, \qquad v = 1.375\sqrt{d}. \qquad (106)$$

From the velocity heads obtained by the above formulas, the velocity curve is plotted. It is rarely found to be a regular mathematical figure, and exact methods cannot, therefore, be employed in obtaining the mean velocity. The following approximate method is sufficiently accurate for all practical purposes.

The diameters of the different rings A, B, C, etc., where pitometer observations were made, are laid off on the diagram, and lines are drawn through them intersecting the velocity curve. The mean velocity in the upper and lower half of each ring is obtained by inspection, and the average of these two numbers is taken as the mean velocity in the ring. By multiplying the mean velocity in each ring by the area of the ring, adding these products, and dividing their sum by the area of the pipe, the mean velocity for the whole pipe is obtained. This mean velocity divided by the center velocity gives the *pipe-coefficient*. If the different rings into which the area of the pipe is divided have the same area, the work of finding the pipe coefficient is, of course, simplified.

When two traverses have been taken on different diameters, the pipe-coefficient is determined by taking the average of the pipe coefficients found for each of the two diameters separately.

513. Interpretation of the Photo-record. — The photographic records taken in the field are developed in the usual manner. The data relating to the record are either written on its book or on a special form which is pasted at one corner on the face of the chart.

A prism photo-record looks, after being developed, somewhat like Fig. 289 and has neither a zero line nor time divisions marked on it. When both legs of the tube are photographed, the height of the pitometer liquid in the *high-pressure* leg of the tube is represented by the line dividing the light and the medium shaded areas, and the height of the liquid in the *low-pressure* leg is shown by the line dividing the medium and dark areas. The ordinate between these lines at any point represents the deflection in the U-tube at that point.

When the deflection increases beyond the range of the prism, so that only one leg of the U-tube, *viz.:* the *low-pressure leg* can be photographed — a condition which obtains throughout when no prism attachment is used — the line of zero deflection must be located on the record before the deflection at any point can be determined. The field notes accompanying the photo-record usually contain sufficient data for determining two points on the zero line, and the remaining points must be obtained by approximation. When the line of zero deflection has been fixed, the ordinate between this line and the deflection curve represents one-half of the actual deflection.

The horizontal lines appearing in a photographic record are marked photographically by notches in the shield, spaced to eliminate instrumental errors. These lines are about $\frac{1}{4}$ in. apart, and each division represents $\frac{1}{2}$ in. deflection when measuring an ordinate from the zero

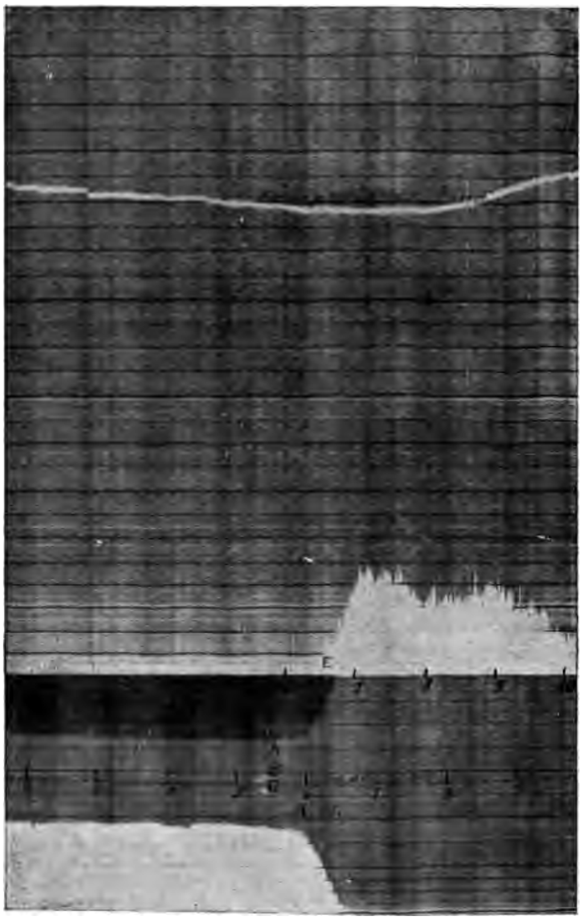


FIG. 289. — Prism Photo-record.

line. The deflections may be readily read from the zero by means of a paper scale, the divisions of which should be taken directly from the chart.

The time divisions are located by proportioning the interval between the time of starting and stopping the record, which is given by the field notes. The time scale for the prism portion of a record is in advance of the time scale above it by an amount which is indicated at the beginning and end of the record. This is due to the fact that for the prism portion of the record the slot of the shield is placed midway between the two legs of the U-tube, while the slot above the prism is placed directly behind one leg of the tube.

After the zero line and time divisions have been marked on the chart (record), the deflection at any point may be ascertained, but the corresponding rates of flow are still to be determined.

The daily rate of flow in any pipe is given by the formula

$$Q = AV \times 7.48 \times 86{,}400, \tag{107}$$

where
Q = discharge in gallons for 24 hours;
A = area of pipe in square feet;
V = mean velocity in feet per second;
7.48 = gallons in a cubic foot;
86,400 = seconds in 24 hours.

The value of V is obtained by the general pitometer formula $V = 2.315\, c \sqrt{d(s-1)}$ (p. 496). By substituting this value in the above equation we obtain

$$Q = 1{,}496{,}100\, Ac\sqrt{d(s-1)}. \tag{108}$$

If we represent all the constants which enter into the above equation by K, the equation is reduced to the simple form:

$$Q = Kc\sqrt{d}. \tag{109}$$

The computation of pitometer measurements are much facilitated by tables giving the values of Kc for different diameters of pipe and different values of c. They may be made by the following simple table by multiplying the value in the last two columns by the traverse coefficient.

TABLE LXVI. — VALUES OF K FOR PITOMETER FORMULAS (109), WITH TRAVERSE COEFFICIENT=100

Size of pipe	Specific gravity of liquid	
	1.25	1.50
4	54.8	77.6
6	123.0	174.0
8	219.0	310.0
10	343.0	485.0
12	494.0	698.0
14	672.0	951.0
16	878.0	1,240.0
18	1110.0	1,570.0
20	1370.0	1,940.0
24	1970.0	2,790.0
30	3080.0	4,360.0
36	4450.0	6,290.0
42	6050.0	8.560.0
48	7900.0	11,200.0

When several records are to be taken at the same pitometer station, it is convenient to prepare a *rate-scale*. The rates of flow corresponding to different deflections are marked on this scale, so that by applying the zero of the scale to the zero line of deflections, the rate of discharge at any given time can be read directly on the scale. As already

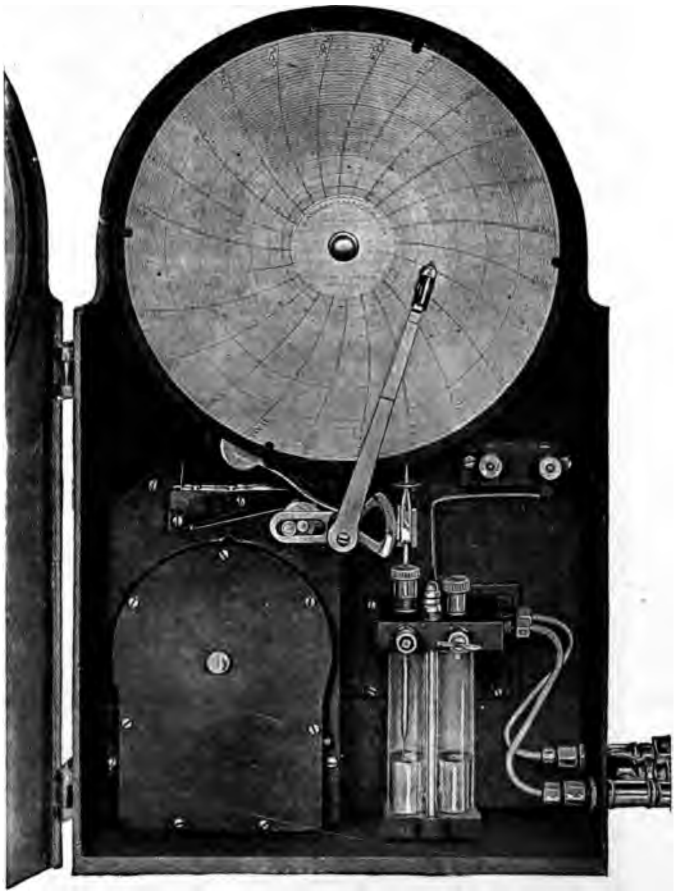


FIG. 290. — Lanham Manograph.

explained, changes in temperature affect the specific gravity of the pitometer liquid and thus cause different rate values for given deflections and corresponding changes in the elevation of the zero-line.

514. The Lanham Manograph * (Fig. 290) is a very accurate and simple form of recording instrument, for use in connection with pitot tubes, for measuring flow of water or gases through mains or conduits.

* Manufactured by the Water Works Equipment Co., New York City.

It was devised a few years ago by Paul Lanham, in charge of water surveys for leakage in Washington, D. C., for use in connection with his work in that city. It is now being very successfully used in Washington, D. C., Baltimore, Md., and other cities.

The great difficulty in making an accurate recorder, which is to be actuated by the small differential pressures that have to be measured in Pitot tube work on mains under pressure, is that a stuffing-box, or its equivalent, must be interposed between the sensitive column of liquid, diaphragm, float, or other device, and the chart. This introduces in the chain of mechanism a variable friction of high value, producing erratic inaccurate results. In the manograph this difficulty is entirely overcome by using electrical means to furnish the connecting link between the sensitive mercury column and the pen mechanism, and by actuating all moving parts with power derived from a small water motor, or other similar device, forming an integral part of the recorder. The charts produced are exact records to a multiplied scale of the changes in level of the mercury in the special form of glass U-tube used. The instrument is provided with a U-tube, filled partly with mercury and partly with water. The top of each leg of the U-tube is connected by a small tube with the top of one of the tubes of a pitometer, which has been inserted into a water main as described on p. 489, or to one of the pressure tubes of a Venturi meter. By this arrangement the mercury in the U-tube will sink in one of the legs and rise in the other, whenever the water in the main is in motion, exactly as in the U-tube of a pitometer. A rod having a fine platinum contact wire at its lower end is made, by suitable connection with the motor, mentioned above, to reciprocate up and down in one of the legs of the U-tube. This rod and wire form part of an electrical circuit which remains closed so long as the wire is in the mercury, but is broken the instant the wire leaves the mercury. The current for this circuit is obtained from a storage battery. By means of an electro-magnet, the pen is kept in contact with the recording sheet, so long as the circuit is closed, but it rises from this sheet the moment the circuit becomes open. By this sensitive arrangement the exact lengths of the lines (ordinates) marked on the sheet are determined.

The level of the mercury column is visible, permitting a check reading to be taken at any time, to determine the accuracy of the instrument, thus eliminating any uncertainty as to the proper operation. The accuracy is permanent throughout the life of the instrument, as there are no calibrated springs, floats, diaphragms, etc., to deteriorate.

The manograph has all the advantages of the simple U-tube manometer, the pen recording feature being added, without the slightest sacrifice of sensitiveness or accuracy. The principle of operation is such that absolutely no work is done by the small differential pressures under measurement beyond that required to balance the mercury in the U-tube.

The instruments are made up in a variety of styles to suit the conditions of operation. Among these are the Type "B," portable instrument, weighing only six pounds; the Type "C," station instrument; the Type "A," special, high-speed instrument for water surveys and tests of short duration; and the Type "S," multiple chart recorder for measurements of flow through a number of mains, such as discharge mains from pumping stations, filter plants, etc.

CHAPTER XXXI

WATER METERS

515. Classification of Water Meters. — Water meters are mechanical contrivances for measuring the quantity of water flowing through an aqueduct, water main, or service-pipe. They can be divided into two general classes, *viz.:* 1. Positive displacement meters, in which the volume of the water is actually measured; 2. Inferential meters, in which the volume of the water is inferred from the velocity of the current, from difference in pressure of the water at certain points of the meter, or from the area of an automatically adjustable opening in the meter through which the water must flow.

According to their mechanical construction, meters may, also, be divided as follows:

A. Positive Displacement Meters

1. Tilting or revolving bucket meters.
2. Reciprocating piston meters.
3. Rotary piston meters.
4. Oscillating piston meters.
5. Nutating piston meters (disc meters).

B. Inferential Meters

6. Impact current meters.
7. Reaction current meters.
8. Differential pressure meters.
9. Variable opening meters.

We may, also, divide meters according to the use to which they are applied:

1. Ordinary service or house meters.
2. Meters for measuring the flow through a water main or aqueduct.

The greatest demand for water meters has been for house meters. Much ingenuity, and a large expenditure of money, have been required to bring these meters to their present state of excellence.

The first meter of which we have any record was invented by Samuel Crossley, who patented in England in 1825 two types of water meters. In the first (Fig. 291) two buckets are balanced on a common axle. Water flows into one of the buckets until it is nearly filled, when the weight of the water tilts and empties the bucket, and places the second bucket in position to be filled. Each time the buckets are tilted, a

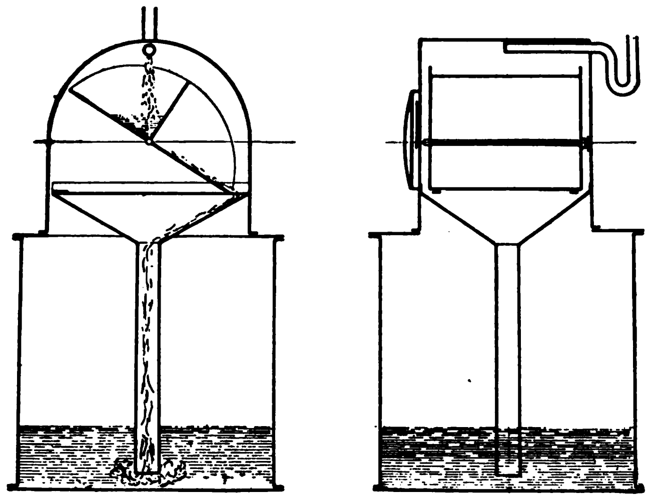


FIG. 291. — Crossley Water Meter with Balanced Buckets.'

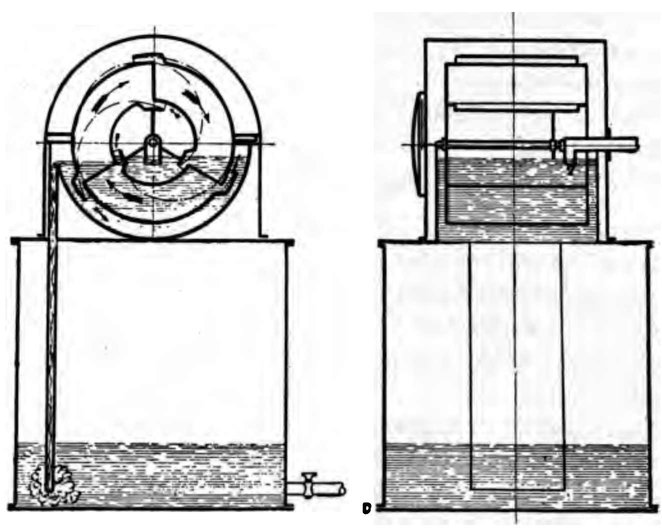


FIG. 291*a*. — Crossley Meter with Bucket Wheel.

definite quantity of water is delivered by the meter. In the type shown in Fig. 291*a* the moving part of the meter consists of a wheel having three buckets, which are filled and discharged in succession, giving a rotary motion to the wheel. For each revolution of the wheel a definite quantity of water is delivered. By a suitable arrangement of gear-wheels, each of the types described above records the quantity of water passing through the meter.

The Crossley meters could not be used for delivering water under pressure and were soon superseded by improved types. From the time Crossley patented his meter up to 1912 more than 1050 patents for water meters have been issued in England and about 1000 in the United States. Most of these patents were granted for improvements in house meters.

The construction of a good meter for this service is by no means easy. Such a meter should register the quantity of water accurately with all flows, from its smallest dribble to full flow, under all possible pressures. It should require very little effective head to operate it, and should cause but a slight loss of pressure in delivering water. The meter should be quiet in action, at all speeds, and free from shocks caused by water-hammer, etc. It should be reliable and free from stoppages, light in weight, simple in construction, permitting ready access to the working parts. The register should be easy to read and should record flows in either direction. The different parts of the meter should be made of materials that will wear well and which will not infect the water, and, finally, the meter should be cheap in first cost and maintenance, and should have a long life.

It is evidently impossible to make a meter satisfy all of the above conditions to the same degree, as some of them tend in opposite directions. Thus, great accuracy may cause increased cost. In selecting a meter, the importance of the different conditions for any given case must be carefully weighed. If the water supplied is costly, it may be advisable to use a dear type of meter, on account of its accuracy, while with water abundant and cheap, some simple kind of meter, lacking in accuracy, may be selected, because it is cheap.

516. Registers. — Two different types of registers are made for most meters, *viz.: circular* and *straight reading* registers. They are shown in Fig. 292. In the circular register, the hands of the different dials are moved by a simple train of gears in constant mesh, each gear-wheel having ten times the number of teeth of the pinion which drives it. In the straight reading registers, the motion of the disc is transmitted through the intermediate gears to a shaft provided with a worm gear, which revolves a horizontal shaft on which *mutilated gears* are placed. Each of these gears has only enough teeth to make one-tenth of a revolution, while the gear on its right side makes a complete revolution. These gears are, therefore, in intermittent mesh. While it is easier to read the straight reading than the circular register, mistakes may occur in reading the former by some of the figures becoming partly discolored or dirty. With the circular registers the correct figure to read in any dial is indicated by the position of the register hand. As the train of gears of a circular register becomes worn, it may happen that the hand of a dial may be slightly behind the position in which it should be, owing to lost motion. The correct number to read in such

CIRCULAR STRAIGHT READING

FIG. 292. — Meter Registers.

a case is indicated by the position of the hand of the next lower dial. If this hand has passed the zero point, the hand of the next higher dial should be on a number of its dial or slightly beyond it. For this reason it is always advisable to read the dials of a circular register from the smallest to the highest. The dial giving fractions of a unit is only used for testing purposes and is disregarded in the regular reading. In the next dial, marked 10, one revolution of the hand indicates 10 units (gallons, cubic feet, etc.). In the circle marked 100 one revolution of the hand indicates 100 units and so on for the other circles; the number of each denotes the units flowing through the meter for each revolution of the hand.

The registers of water meters are made to indicate U. S. gallons, imperial gallons, cubic feet, or many other units of measurement that may be ordered. In commercial practice one cubic foot of water is taken to be equal to 7.5 U. S. gallons.

To avoid errors being made by inexperienced or careless employees in reading water meters, a special meter-book is made by the Pittsburgh Meter Company, in which the person reading the meter marks the position of each of the hands of the dials. This makes it possible to check the reading when it is handed in.

516a. Dial Extensions. — When water meters are set in sidewalks, boxes, or walls, where they cannot be easily read from the surface, a dial extension, like that shown in Fig. 293, is used. It is made in standard lengths of 3 ft. of wrought iron pipe provided with an iron base and top, all thoroughly galvanized. Inside of this casing a phosphor bronze spindle, when attached, acts as an extension to the main driving spindle of the meter.

517. Setting Water Meters. — All water meters should be set in places where they are accessible for inspection or repairs, and where freezing cannot occur. They should not be set more than $3\frac{1}{2}$ ft. above the floor and should be provided with suitable couplings to make it easy to disconnect them from the service-pipes. A stop-cock should be placed on each side of the meter. Meters, 1 in. or more in diameter, which may be tested on the premises, should have a test-tee and check-valve on the outlet side of the meter, between the meter and the stop-cock, and it is, also, advisable to place a relief-valve on the outlet side of the meter.

No red or white lead should be used in making the joints, as some of it might get into the meter. In starting the meter, the water should be turned on very slowly until the working parts are in equilibrium and the air has been exhausted from the meter.

517a. Fish-traps. — When the water carries much sand, or or-

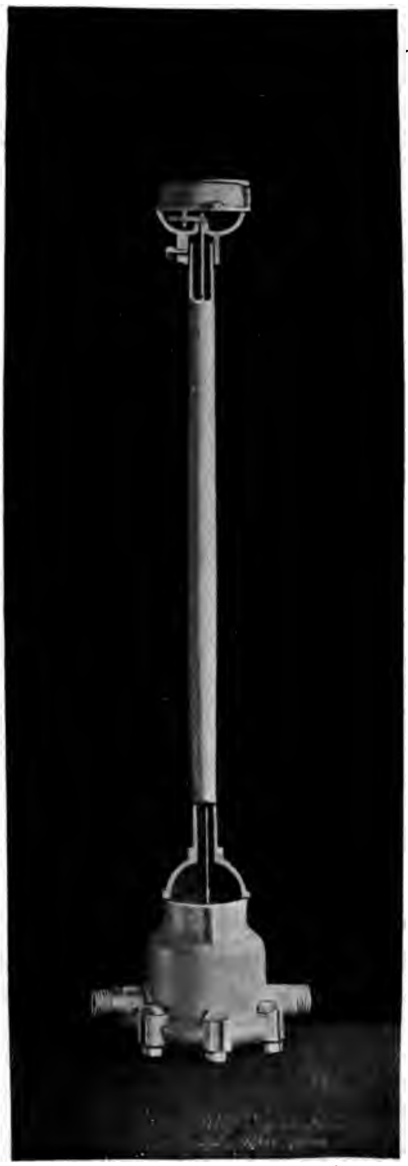


Fig. 293. — Extension Dial.

Fig. 293*a*. — Trident Fish-trap.

ganic matter or other materials that may clog the meter, it is advisable to insert a *fish-trap* in the service-pipe. Fig. 293*a* shows such a trap made by the Neptune Meter Company. It consists of a large cast iron chamber provided with heavy cast iron strainers. The sediment, etc., coming through the service-pipe is intercepted by the trap and must be periodically removed.

518. Description of Water Meters. — More ingenuity has been spent in inventing and improving water meters than has been devoted to any other device connected with water works. For measuring water within a limit of about 2 per cent, plus or minus, some type of piston meter must be used; but where water is cheap and an error of registering of 5 per cent, plus or minus can be permitted, preference is given to current meters, on account of their cheapness. In the following pages we shall describe the principal water meters used in America and in Europe.

American Water Meters

519. Piston Meters. — The first type of water meter that was put into actual use is the reciprocating piston meter. It consists of a water-tight piston which is moved up and down in a cylinder by the water flowing through the meter. For each stroke a fixed quantity of water flows through the meter. The motion of the piston actuates a suitable registering mechanism. Instead of one cylinder, two or more may be used. While accurate, piston meters are costly, heavy, and difficult to keep water-tight. They are used largely in England, Belgium, and France, but have been almost entirely replaced in America for ordinary service by rotary-piston, disc-piston, and current meters. We describe below the only reciprocating piston meter manufactured in the United States, and, also, the principal kinds of rotary and disc meters.

520. *The Worthington Duplex Piston Meter.* — Henry Worthington, of New York, and his successors have manufactured piston meters for over fifty years. The latest form of these meters is the duplex piston meter (Fig. 294) which is similar to the duplex pump made by the same manufacturers. The design of the latter was really based upon the construction of the former. In the duplex meter, two horizontal cylinders are placed side by side. Each has a plunger which moves forward and backward through bronze linings, carrying a slide-valve over ports in the bottom of the meter. Through these ports, the cylinder on each side of the plunger is alternately connected with the inlet and outlet of the meter. The register is operated by a lever, which receives a reciprocating motion from one of the plungers and actuates a spindle and ratchet. By this arrangement the dial pointers of the register are moved once for every four strokes of the plungers.

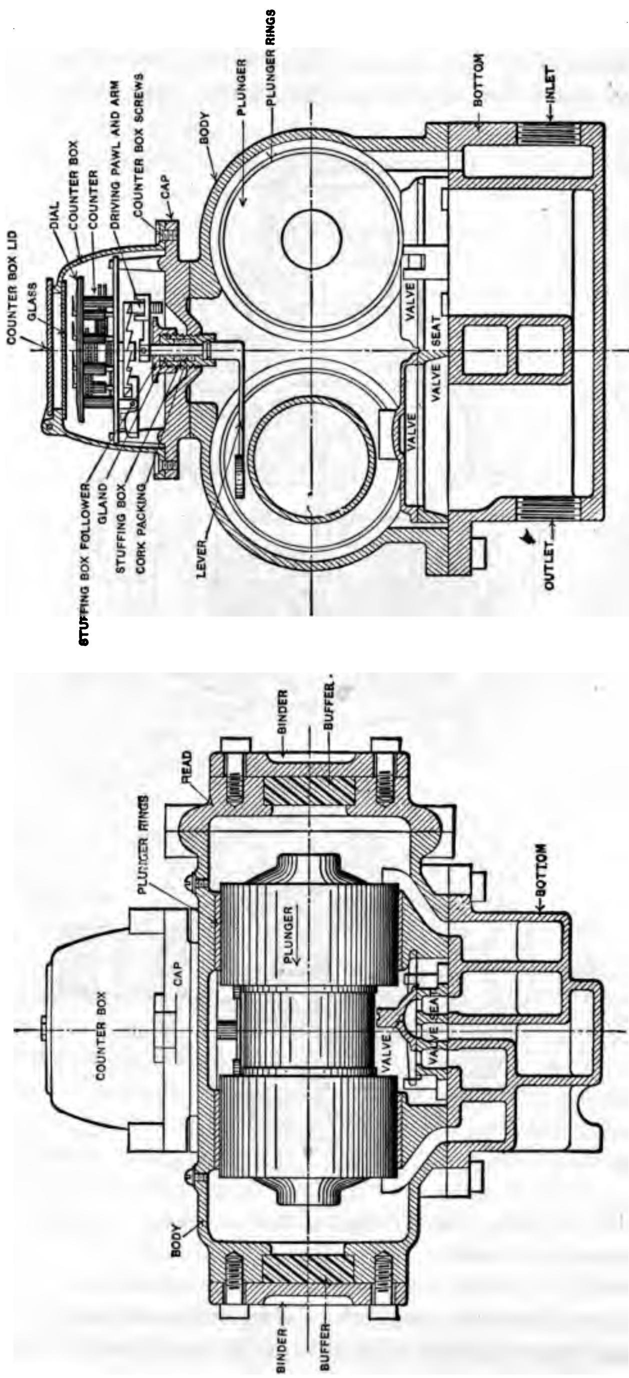


FIG. 294. — Worthington Duplex Piston Meter.

As all water passing through the meter must enter one of the cylinders and be displaced by the plunger, the register records accurately the quantity of water flowing through the meter. An over-registration is

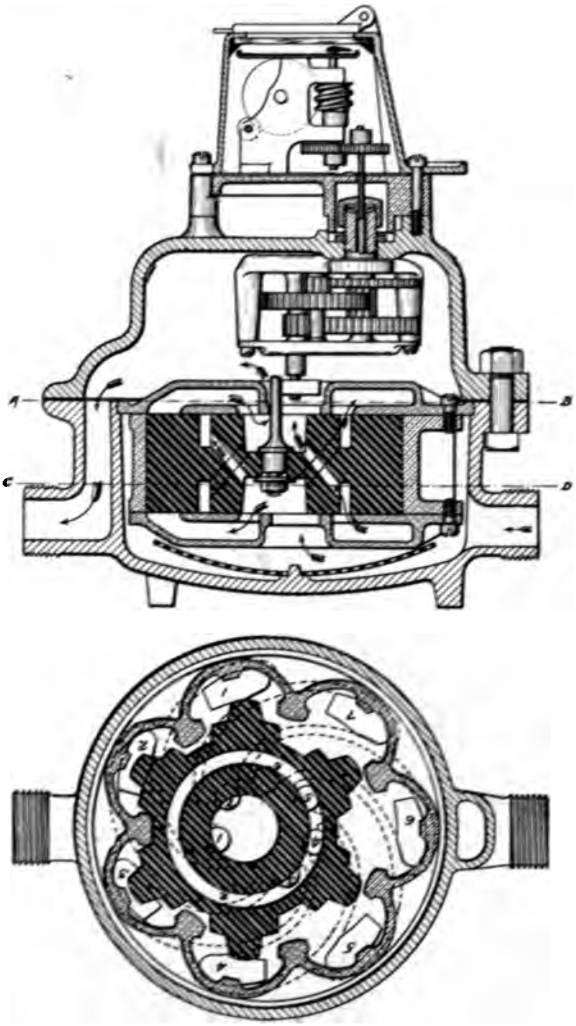


Fig. 295. — Crown Water Meter.

not possible, because the plungers cannot move without displacing the water in the cylinders.

The cylinders, bottom, cap, and heads are made of cast iron. The other parts of the meter are made of a special brass composition. No intermediate train is required for this meter, owing to the large volume of water displaced for each movement of the lever that operates the register.

This meter is especially adapted for large and heavy waterworks service, and is made for pressures up to 175 lbs. per square inch. It is not affected by hot water, which sometimes backs up in service-pipes.

521. *The Crown Meter* * (Figs. 295 and 296), invented and patented by Lewis H. Nash in 1879, is the first rotary piston displacement meter manufactured for measuring water. A gear-wheel with internal teeth, called the crown, is fitted closely in a casing, having an inlet and an outlet spud. The rotary piston is provided with teeth and acts as an internal gear in the crown. A *top-head* and a *bottom-head* fit water-tight upon the piston. The space between the two heads forms the measuring chamber. It is divided into a receiving and a discharging chamber by the piston, which fits so closely in the crown as to form practically a water-tight partition between the two divisions of the measuring chamber. As the piston is revolved by the water flowing through the meter, the positions of the receiving and the discharging chambers are constantly changing.

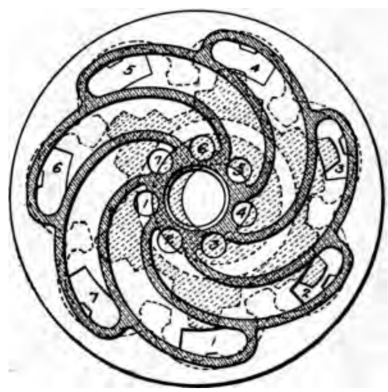


FIG. 296. — Parts of Crown Water Meter.

The water, after passing through a screen, placed in the bottom of the casing, flows through a central hole in the bottom-head into the measuring chamber and, after revolving the piston, is discharged from the measuring chamber through a central hole in the top-head. Both heads have a number of passages or ports, each of which begins with a small opening near the central hole of the head and terminates in a segmental hole between two consecutive teeth of the crown. The number of these ports in each head is one less than the number of teeth in the crown. Each port serves, alternately, to let water into and out of the measuring chamber. The top and the bottom of the piston act as rotary slide-valves, respectively, for the ports in the top- and the bottom-heads, connecting each port, alternately, with the inlet and outlet of the measuring chamber. To enable the piston to act in this manner, a central and, also, an annular recess, are cut both in the top and the bottom of the piston. Each of the central recesses is connected

* Manufactured by National Meter Co., New York.

with the annular recess on the opposite side of the piston by small holes. There are as many of these holes as the piston has teeth.

The manner in which the ports are, alternately, connected with the inlet and the outlet of the measuring chamber is seen in Fig. 296, which shows sections of a meter having seven ports. Three ports (1, 2, and 7) are open, admitting water to the measuring chamber, while three other ports (3, 4, and 5) are discharging water into the outlet. One

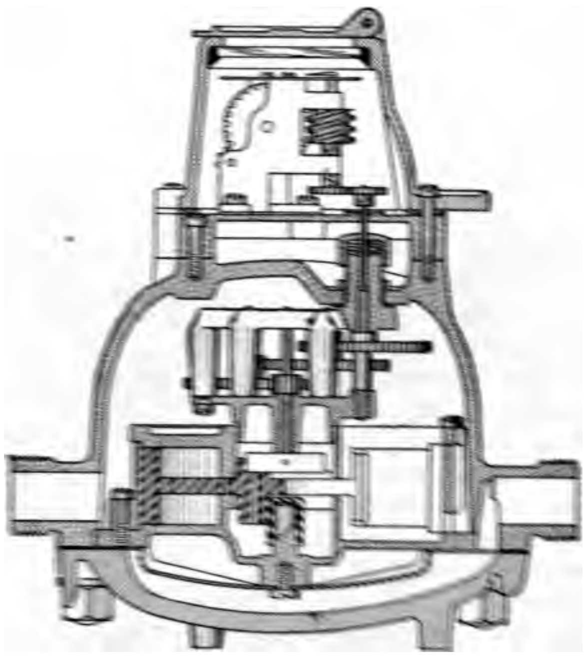


FIG. 297. — Empire Water Meter.

port (6) is closed. As the piston revolves, each port serves, alternately, as an inlet and an outlet passage.

The casing, crown, and heads are made of bronze and the piston is of hard rubber to give it a specific gravity but slightly in excess of that of water. The meter is very sensitive, and remains accurate for a long time, as the large openings in the measuring chamber will hold sediment, etc., that passes through the screen. When the top or bottom of the piston wears off appreciably, water-tightness can easily be restored by grinding down [illegible]

The National Met[illegible] was the first concern to manufacture [illegible] these meters, made 25 years ago, [illegible] by this company have one tooth less than the crown. This causes a constant change of the teeth which engage with each other and produces a uniform wear.

The Hersey Manufacturing Company, of Boston, Mass., makes a crown meter, not called, however, by that name, which differs from the one described above in the form of the tooth, and in the fact that the

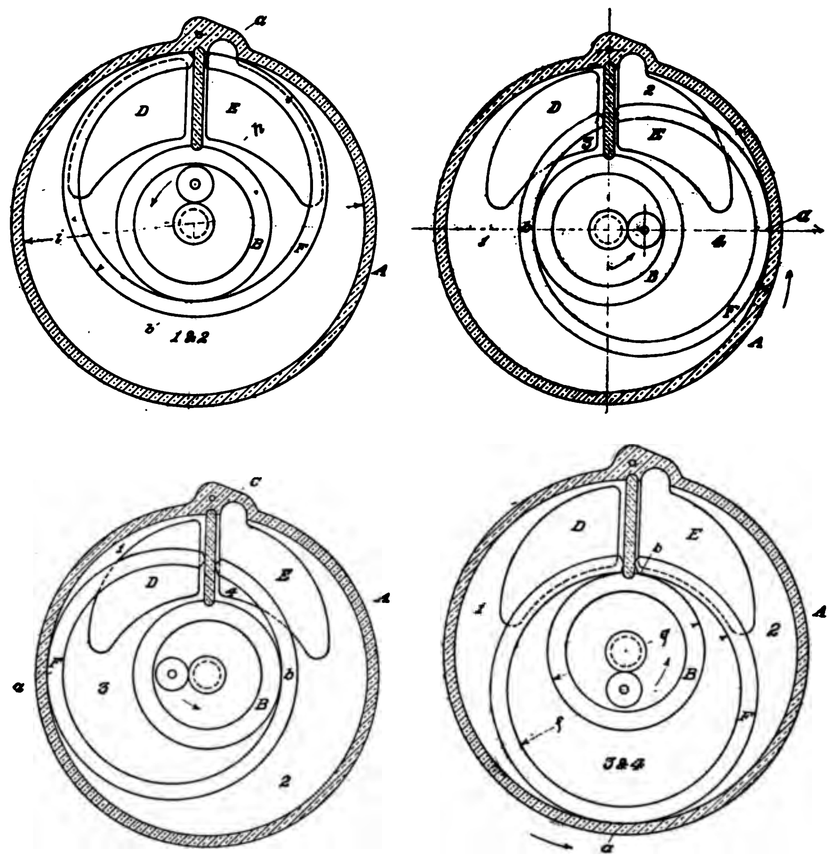


FIG. 298. — Portions of Piston of Empire Water Meter.

piston and crown are given the same number of teeth. With this arrangement each point of the piston follows a circular path, which remains constant.

522. *The Empire Meter* * was invented and patented by Lewis H. Nash in 1884. It belongs to the class of positive displacement meters. The manner in which it acts will readily be understood from Figs. 297 and 298. In a cylindrical casing *A*, a piston consisting of a split ring

* Manufactured by National Meter Co., New York.

of cylindrical form is placed. A diaphragm attached to the inside of the casing fits into the slot of the piston. At the center of the casing, the hollow cylindrical abutment *B* is placed, and is provided at its center with a roller revolving on a pin. A pin is fastened to the piston at its axis, and fits closely between the inside of the abutment and the roller. This pin guides the piston in its motion, and prevents it from bearing hard against the casing. The path this pin describes is circular, but the motion of the piston itself consists of oscillations from one side of the casing to the other, forwards and backwards. Suitable heads close the top and bottom of the casing and fit water-tight upon the piston. The inlet and outlet ports are placed in the two heads, the inlet ports of the two heads being connected bv a passage in the casing.

The water enters by port *D* and leaves by port *E*. The piston and the diaphragm divide the space between the two heads of the casing into two receiving and two discharging chambers, which are constantly changing in size as the piston oscillates. Two of these chambers are inside and the others outside of the pistons. The water flows in a steady stream through the measuring chamber, causing the piston to oscillate as described above. For each complete oscillation, a definite quantity of water is measured. The motion of the piston is communicated in the usual way, through an intermediate gear-train to the registering device.

The casing with its piston is placed in a suitable casting, the cover of which has the inlet- and the outlet-spuds. A screen is provided at the bottom of the casing, to interrupt sediment, etc. The main casting, the casing, and its covers are made of bronze, but the piston is made of hard rubber, to obtain as much lightness as possible. Half-way between its ends, a perforated web, to which the guide pin is attached, is placed. The abutment is cast in the two heads and fits water-tight on the web of the piston. Each head is provided with inlet- and outlet-ports.

The Empire Meter is exceedingly sensitive. It is simple in construction, and any wearing of the ends of the piston can be remedied by grinding down the casing. If the surface of the piston has been worn off, the substitution of a slightly larger roller pin makes the piston again water-tight.

523. Disc Meters. — In 1830 the brothers Dakeye, of England, invented a new type of steam engine, in which they substituted for the usual steam piston a circular disc, supported on a spherical bearing.

Frank Lambert patented in the United States in 1884 a typewriter in which the disc action is used as a multiple key. In 1887 James

Davies, of England, invented the first disc water meter, which was similar in many respects to the disc steam engine. Two years later Davies patented this device in the United States. This meter proved to be a failure, mainly because it had only one casing, which was subjected to the full pressure of the water. Any distortion of this casing stopped the disc from oscillation.

Lewis H. Nash designed the first disc meter in the United States, and obtained a patent for this invention on March 20, 1888. John Thomson and the above-mentioned Frank Lambert invented jointly certain improvements in oscillating disc water meters, for which they obtained a United States patent, soon after Mr. Nash obtained his patent. The most important of these improvements consisted in providing a special, inner casing, or measuring chamber, for the disc. The manufacture of these meters, called the Thomson meter, was begun in January, 1889, by the Water Waste Prevention Company. This concern was reorganized as the Thomson Meter Company, and is still extensively engaged in the manufacture and sale of disc meters. With the improvements made, the meter now manufactured by this company is known as the *Lambert Meter*. In this meter the disc is made flat, while in the design patented by Lewis H. Nash it is conically shaped.

On account of the simplicity of its construction, its durability, cheapness, and the small loss of head it occasions, the disc meter was soon favorably received, and it is now more extensively used in America for services of $\frac{1}{2}$ to 6 ins. than any other type of meter.

524. *The Lambert Disc Meter* * (Fig. 299) consists of the following principal parts: (1) The main casting, having the inlet- and outlet-spuds; (2) an inner casing, forming the measuring chamber; (3) the disc-piston, which is placed in the inner casing; (4) the intermediate gear-train; and (5) the registering device.

The main casting and the casing are made of bronze, in halves which are bolted together. The casing fits closely in the main casting. This clearance serves the purpose of a screen, and permits only very small particles of sediment, etc., to get into the measuring chamber. The volume which the disc generates by its oscillation forms the inside of the casing. It consists of a spherical socket at the center; two frustums of cones, with their small ends towards each other; and a spherical zone at the perimeter of the casing. The disc remains always in contact with the top and bottom cones, and its edge clears the spherical zone by only about $\frac{3}{1000}$ of an inch, so as to make a practically water-tight joint. Two inlet-ports and one outlet-port let the

* Manufactured by Thomson Meter Co., Brooklyn, N. Y.

water, respectively, into and out of the measuring chamber, both from above and below the disc. A radial diaphragm placed between the inlet- and outlet-ports, together with the disc, which is provided with a slot that fits over the diaphragm, divides the inside of the casing into two chambers above and two below the disc. Those to the right of the

Fig. 299. — Lambert Disc Meter.

disc receive the water, while the other two discharge it. At one particular position of the disc the two lower chambers merge into one and the same occurs for another position with the two top chambers.

The disc is made of hard rubber and is reinforced by an inner, perforated steel plate (Fig. 300) to prevent it from breaking. This plate is held firmly in the disc by the rubber filling the perforations, which becomes hard by the vulcanizing. The central ball-bearing of the disc (Fig. 301) is made in halves which are bolted together by the spindle of the disc.

A conical rubber roller is attached to the top of the disc-spindle. It bears against a conical tube, which forms the journal of the first spindle of the intermediate gear. The latter spindle has, attached to its lower end, a two-arm lever, which is turned by the roller of the disc-spindle, as the disc oscillates.

The intermediate gear-train consists of four pinions and four gear

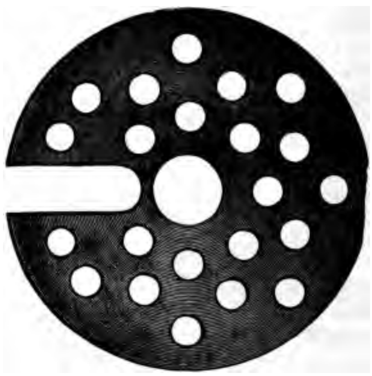


FIG. 300. — Disc.

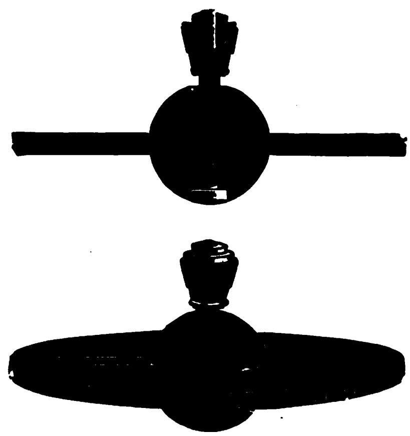


FIG. 301. — Central Ball-bearing.

wheels. The former are made of the hardest grade of phosphor bronze, and the latter of semi-hard bronze composition. All of the spindles of this train are made of the hardest wire-drawn phosphor bronze, and turn freely in composition bearings.

The register records, by a suitable arrangement of pinions, gear wheels, etc., the quantity of water flowing through the meter either on a number of dials or by a self-reading counter (p. 506). In the former case the hands of all dials are made to turn in the same direction as those of a clock, so as to avoid errors in reading the meter.

The inlet- and outlet-spuds of the $\frac{5}{8}$-, $\frac{3}{4}$-, and 1-in. meters have male threads; those of the $1\frac{1}{4}$-, $1\frac{1}{2}$-, and 2-inch sizes have female threads. All meters above the 2-in. size are connected to the service-pipes by flanges and bolts.

525. *The Trident Disc Meter* * (Fig. 302) consists of four self-contained units which can be easily assembled: The main casting — or housing — having the inlet- and outlet-spuds; the measuring chamber, containing the disc piston; the gear-train, and the register.

For localities where freezing may occur, the housing is a heavy, hollow bronze casting made in one piece, the bottom being closed by

* Manufactured by Neptune Meter Co., New York.

a cast iron, brass-lined cap, and the top being covered by the register casing. The bottom cap has four arms which are slotted out to facilitate the placing of the bolts which attach its cap to the housing. These bolts have T heads which fit closely to the main casting and prevent

Fig. 302. — Trident Disc Meter.

the bolts from turning. The arms of the bottom cap are calculated to break under a pressure of about 400 lbs. per square inch. By this simple arrangement, the damage resulting from freezing is limited to the breaking of the bottom cap, which can be replaced at a trifling cost. Where there is no danger of freezing, the *breakable bottom* is omitted and the housing made in two parts, forming what is known as *a split case.*

Fig. 303. — Disc, Diaphragm, and Antifriction Roller *R*.

By removing four bolts, the upper part of the casing, and all the moving parts of the meter, can be removed without disconnecting the meter from the service-pipe. The lower part of the casing can be closed by a cap and service can be continued while the meter is being repaired.

The measuring chamber of the meter is made of bronze in two halves, which are pressed together after the piston has been inserted and are then placed in the casing. The disc-piston and its spherical center are made of hard rubber. The wear of the edge of the disc is prevented by setting in the disc an anti-friction roller (Fig. 303) which rotates in a recess in the wall of the measuring chamber, the wearing surface being liberally proportioned to sustain the pressure.

The gear-train, complete in itself, is held in the casing by a single unit. The pinions and gears are made of compositions that resist the chemical action of the hardest water, and are cut accurately by automatic machinery. The train contains only eight parts, each of which can be duplicated.

The register is made of non-corrosive materials and the number of stock parts has been reduced to about a minimum. The top and bottom plates are fitted with hard rubber bushings for all the spindles. A bearing is thus provided which is non-corrosive, and which reduces friction to a minimum. The registers are made circular, or straight reading, as desired. The testing hand revolves in a circle, having almost the diameter of the dial plate. This facilitates close reading during tests.

526. Manufacturers of Disc Meters. — Almost every manufacturer of water meters in the United States makes at least one style of disc meter. While based upon the same general principle, the different makes of disc meters vary in the details of construction and materials.

The greatest trouble experienced with the early disc meters was the breaking of the disc — which is made of vulcanized rubber, to give it lightness — under continuous high speed. Various expedients have been adopted to remedy this defect.

The names of the manufacturers of disc meters in the United States and the trade names of the meters are given in the following table:

TABLE LXVIII. — AMERICAN DISC METERS

Trade name of meter	Manufacturer
Lambert	Thomson Meter Co., Brooklyn, N. Y.
Trident	Neptune Meter Co., New York
Keystone	Pittsburgh Meter Co., Pittsburgh, Pa.
Worthington	Henry R. Worthington
Nash disc	National Meter Co., New York
Hersey	Hersey Meter Co., Boston, Mass.
King	Union Meter Co., Worcester, Mass.
Watch dog	Gannon Meter Co., Newark, N. J.
Badger	Badger Meter Co., Milwaukee, Wis.
American *	Buffalo Meter Co., Buffalo, N. Y.
Niagara *	Buffalo Meter Co., Buffalo, N. Y.

* The American and Niagara meters are made according to the same model, and differ only in the metal used for the outer casing.

527. Current meters belong to the class of *inferential meters*. They indicate the volume of water flowing through them by measuring its velocity. Good meters of this type will measure the volume of water correctly within 2 to 5 per cent, according to circumstances. Owing to their cheapness, these meters are used where the cost of water is low. Some of the principal American current meters are described below.

528. *The Gem Meter* * (Fig. 304), introduced in 1870, has a hard rubber spiral propeller which is placed in a case-chamber having a slightly larger diameter. The propeller is mounted on spindles, the

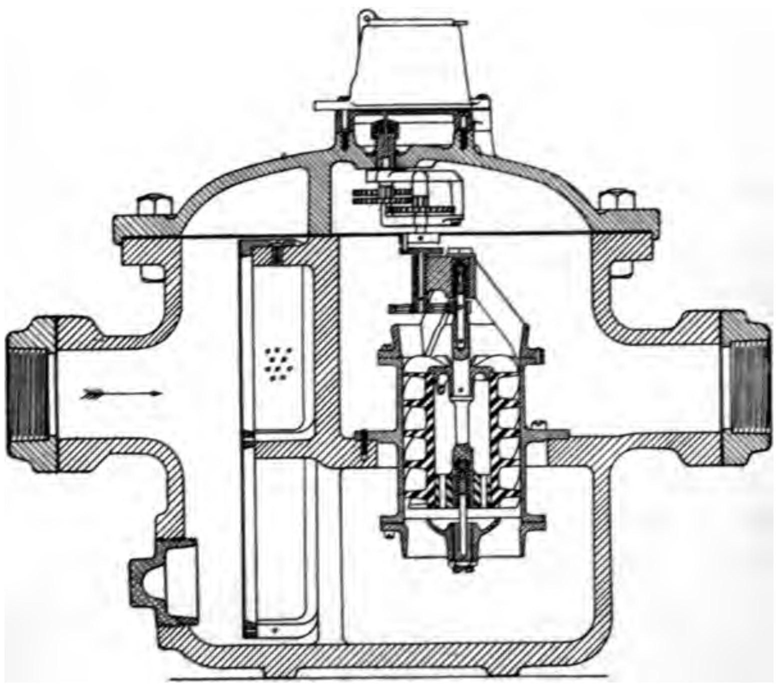


FIG. 304. — Gem Water Meter.

thrust of which is taken by hardened steel points, which are placed in the end of bearings, where no injury from water is possible. The water, after passing through a screen, flows upward through the chamber of the propeller, revolving the latter, in proportion to the pitch of its blades. The meter thus indicates the velocity of the water, from which the quantities passing through the propeller chamber can be inferred. The revolutions of the propeller are transmitted to the dial by suitable mechanism.

* Manufactured by National Meter Co., New York.

529. *The Nilo Meter* * (Fig. 305) measures the flow of the water by means of a hard rubber piston revolving at a speed proportional to the velocity of the flow. The piston has two sections, the upper one having right-hand, and the lower one, left-hand helical vanes. A deflector-plate separates the two halves of the piston, and insures equal discharges of water from both halves of the piston, and maintains an equality of thrust. The vanes are encased in a rubber shell which protects them against breakage. The piston is mounted on a bronze shaft which has a jewelled bearing.

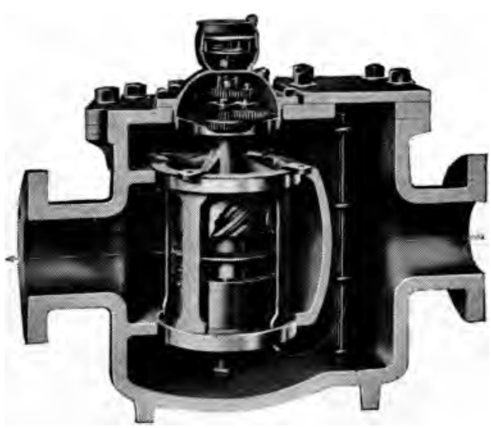


FIG. 305. — Nilo Water Meter.

The motion of the piston is transmitted through an intermediate gear-train to the register. The gear-train is placed in a dome, which can be turned around so as to facilitate the reading of the register. A screen of the bar type is placed at the inlet. The water, as it enters the meter, is divided into two equal streams, and flows respectively downward and upward through the piston chamber, and is deflected to the outlet by the deflector mentioned above.

530. *The Trident Crest Meter* † (Figs. 306 and 307) consists of a main casting having an inlet- and an outlet-spud, the measuring device, the gear-train and the register. The main casting has, at the bottom, a blow-off opening provided with a cap, and is closed at the top by a bronze cover to which the gear-train and register are attached.

The measuring device consists of two hard rubber propellers, one a right-hand and the other a left-hand helix, mounted on a vertical spindle of phosphor bronze in a bronze casing, which rests on an accurately machined seat in the main casting. Each propeller is placed in a water-tight, hollow chamber in the casing, and is protected by a cage that serves as a strainer. The spindle is carried on an agate bearing, and revolves in hard rubber bushings, which keep the propellers concentric with their housing. It is connected loosely, but positively, with the driving pinion of the gear-train, which is placed in a recess in the bottom of the bronze cover of the main casting. The register is attached to the top of this cover.

* Manufactured by Union Water Meter Co., Worcester, Mass.

† Manufactured by Neptune Meter Co., New York.

As the water enters the meter, it is divided by a projection in the main casting into two equal streams, which flow with the same velocity through the two propellers, revolving them. The two streams of water react upon each other, radiate in all directions, and flow, at right angles to the line of impact, through ports to a large chamber, just back of the outlet-spud. As the weight of the propellers with their

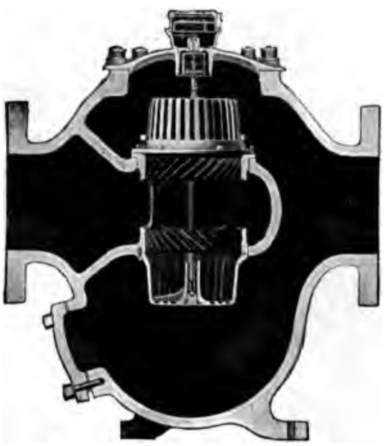


Fig. 306.

Fig. 307.

Trident Crest Meter.

spindle is but slightly greater than the water they displace, and as the action of one propeller balances that of the other, there is practically no weight or thrust upon the agate bearing. Friction, and consequently wear, is thus reduced to a minimum.

The water passages and chambers are all made of ample dimensions so as to avoid loss of head. The restriction caused by the propeller spaces can be increased or diminished, according to the desired degree of sensibility. All the moving parts of the meter can readily be taken out through the opening at the top of the main casting, without disconnecting the meter from the service-pipe. For *high duty service*, the crest meter is made extra strong.

Placed in a housing especially adapted for the purpose, the Trident crest meter is used as a stand-pipe and water cart meter. For this purpose it is made in two sizes ($1\frac{1}{2}$ and 2 ins.). The inlet opening is made at the bottom and the delivery at one side. The water cart meter is provided with lugs for bolting it directly to the wood work of the tank.

531. *The Turbine Meter* * (Fig. 308) is a development of the well-known Worthington turbine pump. It measures the velocity of the

* Manufactured by Henry R. Worthington, New York.

water passing through the meter by means of a double turbine wheel, having two sets of vanes, one right-hand and the other left-hand. This wheel is placed in the main casting of the meter in a chamber of the volute pattern, which provides at all points of the circumference the exact cross-sectional area required to pass the volume of water discharged by the wheel, without reducing the velocity of the water

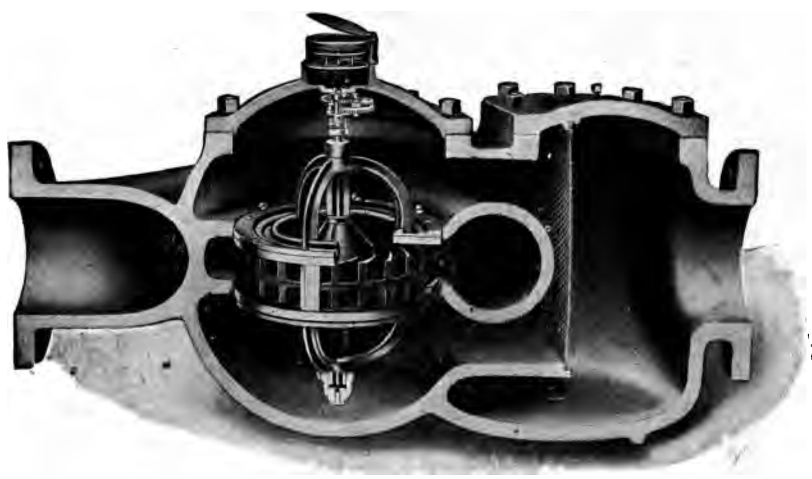


FIG. 308. — Turbine Water Meter.

column. The turbine wheel is made of hard rubber, so as to weigh but a trifle more than the same volume of water. It is mounted on a vertical shaft of bronze, having a jeweled bearing at its lower end.

The water entering the meter passes first through a sliding screen, and is then divided by the wheel chamber into two equal streams, one of which flows downward through the turbine, while the other flows upward. By this arrangement the pressure on the turbine wheel is balanced and friction is reduced to a minimum.

The motion of the wheel is communicated through an intermediate gear to the register. The meter is provided with two hand holes, closed by suitable caps. Through one the screen can be removed, while the turbine wheel with its shaft can be taken out through the other. The meter can, therefore, be easily repaired without disconnecting it from the service-pipe.

Turbine meters are made in sizes of 2 to 12 ins. While principally designed for measuring large flows of water with a minimum loss of pressure, they are, also, sensitive to small flows.

532. *The Eureka Meter* * (Fig. 309) measures the velocity of the water by means of a wing-wheel. The water entering the meter flows first through a strainer, and then through ports, located diametrically opposite to each other, into a compartment that surrounds the measuring chamber. From this compartment it passes through tangential ports, spaced equally around the walls of the chamber, into the measuring chambers, where its impact rotates the vanes of the wing-wheel in a horizontal plane at a speed that is proportional to the velocity of the water. The motion of the wheel is communicated through a

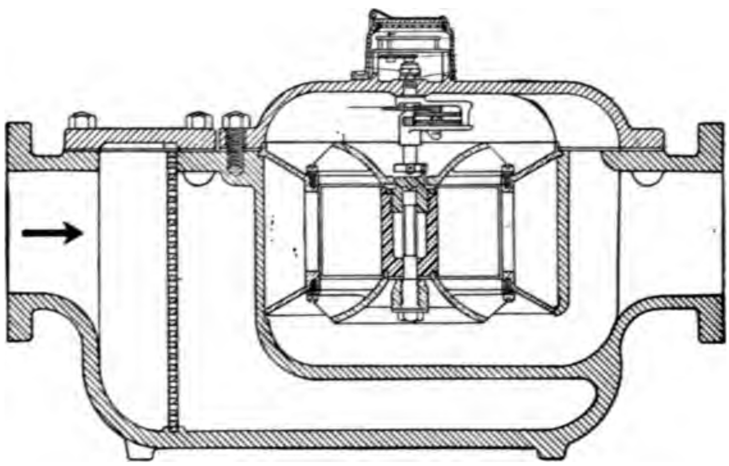


FIG. 309. — Eureka Water Meter.

crank on the top of the wheel to an intermediate gear, and from this to the register placed on top of the meter. After its contact with the vanes of the wheel, the water divides into two equal streams which leave the measuring chamber through outlet-ports provided in the top and bottom plates of the measuring chamber, and thus reaches the outlet passage of the meter.

The wing-wheel is made of hard rubber and is mounted on bearings which are designed to minimize the friction. The inlet-ports to the measuring chamber direct the inflowing water against the blades of the wheel in a manner which increases the meter's sensibility.

The strainer is placed sufficiently far from the inlet of the meter to form a fish-trap for intercepting foreign matter carried by the water.

* Manufactured by Pittsburgh Meter Co., East Pittsburgh, Pa.

533. *The Torrent Meter* * (Fig. 310). — The velocity of the water passing through the meter is measured by a horizontal piston with curved vanes, shaped like a water wheel, which is revolved by the impact of the water. This piston is made of vulcanized hard rubber, and is mounted on a spindle of phosphor bronze, with a vulcanized hard rubber, removable bearing. It is placed in a bronze measuring chamber, which can be removed from the outer casing, which is made of galvanized cast iron. This casing is provided at the inlet opening

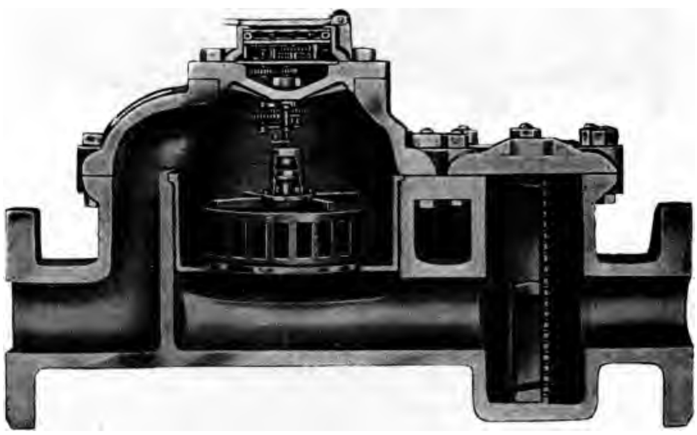


FIG. 310. — Hersey Torrent Meter

with a strainer or fish-trap, which can be cleaned, without disturbing the working parts of the meter.

The water entering the meter passes first through the strainer, and then flows upwards into the wheel, where a deflector turns it into a horizontal direction to the outlet-ports, thus avoiding a vertical thrust through the moving parts. The motion of the piston is transmitted to the register by an intermediate train.

534. Proportional Meters. — In this class of meters, a large volume of water, flowing at a relatively high velocity, is measured by a disc meter placed in a by-pass. Within certain practical limits, the flow of water through the by-pass will be a fixed percentage of that through the main pipe. By a suitable arrangement of gearing, the register of the disc meter is made to indicate the quantity of water flowing through the main pipe.

535. *The Hersey Proportional Meter* * (Fig. 311) is the only meter of this class that is manufactured. Model *PM* meter consists of a

* Made by the Hersey Mfg. Co., South Boston, Mass.

piece of flanged galvanized cast iron pipe, of the same diameter as the water main, except at two points, where the diameter is slightly reduced by two friction-rings. Model P meter has only one friction ring. By means of these rings a portion of the water passing through the main is diverted into a by-pass, in which an ordinary disc meter is

Fig. [illegible] —[illegible] Proportional Meter.

placed. The register of this meter indicates the flow through the main pipe.

These meters are made in sizes of [illegible] to [illegible] ins. diameter, according to model [illegible] and from [illegible] to [illegible] ins. according to model [illegible]. For model [illegible] the disc meter is placed [illegible] in the [illegible], forming the main [illegible] of the proportional meter, and the by-pass is formed in this [illegible] by suitable [illegible]. In model [illegible], the by-pass is formed of two pieces of flanged galvanized cast iron pipe, and is placed [illegible] the main pipe. [illegible] gate [illegible] on each side of the disc meter, [illegible] possible to remove the meter without [illegible] the flow in the main pipe. [illegible] the disc meter [illegible] the water through the meter [illegible] compared, when the [illegible] gate is closed. By [illegible] the water [illegible] the accuracy of the meter [illegible].

[illegible] Meters. [illegible]

537. *The Hersey Detector Meter* * (Fig. 312) is especially designed for fire service, or for combined manufacturing and fire services, in which the meter must register low rates of flow for general use, and high rates of flow in case of fires, without obstructing the flow or causing much loss of pressure. The meter is a combination of model *P* proportional meter with a disc meter, placed in a horizontal by-pass at one side of the main pipe, and has, in addition, an automatic check-valve, which opens only for large flows. For ordinary use, this valve remains closed, and the water flows through the horizontal by-pass, and is registered by its disc meter. For large flows, the check-valve opens automatically, as described below, and the water has practically an unobstructed passage through the main pipe, except at the *friction-ring*, which causes

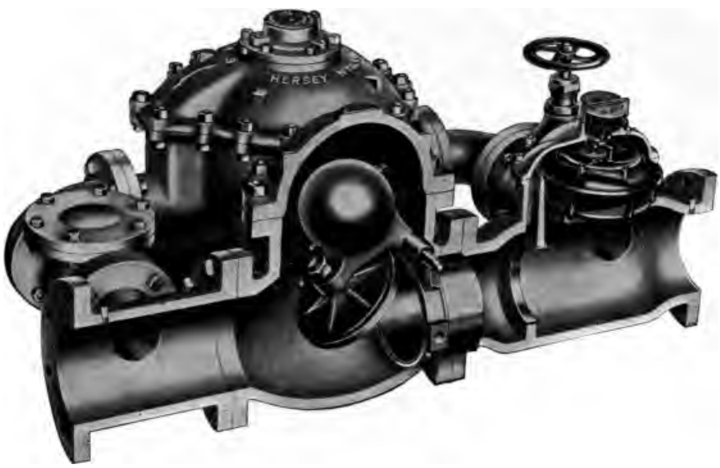


FIG. 312. — Hersey Detector Meter.

a certain proportion of the flow to pass upward through a second by-pass, provided on top of the main pipe, in front of the friction ring. A disc meter, placed in this by-pass, has its register arranged to indicate the total flow of water passing through the main pipe. As soon as the flow is sufficiently reduced, the check-valve closes automatically, and the water passes again through the horizontal by-pass. The combined readings of the two disc meters gives the total quantity of water passing through the meter.

The check-valve which controls the flow through the main pipe has a grooved seat which is open to the atmosphere, thus creating an initial resistance or back pressure equal to about six per cent of the static pressure. Leakage through the atmospheric opening is pre-

* Made by the Hersey Mfg. Co., South Boston, Mass.

vented by a small auxiliary valve, which is located in the grooved seat, and which is operated by the automatic check-valve, so as to be open when the check-valve is closed and vice versa.

When the flow through the horizontal by-pass is sufficiently large to reduce the pressure on the outlet side of the check-valve six per cent, this valve opens automatically. A hollow brass ball, weighted with shot, and attached to the check-valve, makes this valve close quickly, when the flow through the meter has been sufficiently reduced.

The manufacturers claim that this meter is accurate within 5 per cent for all flows, and that it has a very low friction loss, which amounts in a 6-in. meter to only 1 lb. for a flow of 1000 gals. per minute.

538. *The Trident Protectus Meter* * (Fig. 313) consists of three units: a pipe with a sensitive check-valve, which opens for large flows

FIG. 313. — Trident Protectus Meter.

of water, and closes for small flows; a by-pass on one side of the pipe, for the passage of small flows; and a by-pass on the other side of the pipe, which forms with the main pipe a proportional meter. The first-mentioned by-pass is provided with a Trident disc meter for measuring small flows, while the other by-pass has a Trident Crest Meter for measuring the proportion of a large stream of water which passes through the meter.

The check-valve is adjusted to open quickly when the flow reaches a fixed rate of discharge and to close the moment the rate of flow becomes less.

539. Compound meters consist of combinations of current meters with positive displacement meters, the former measuring large flows, while the latter indicate the small flows. The positive displacement meter, which is usually of the disc type, is placed in a by-pass. A con-

* Manufactured by the Neptune Meter Co., New York.

trolling-valve is installed in the main pipe, and is so constructed that it remains closed for small flows and opens automatically for large flows. The efficiency of a compound meter depends entirely upon the controlling-valve.

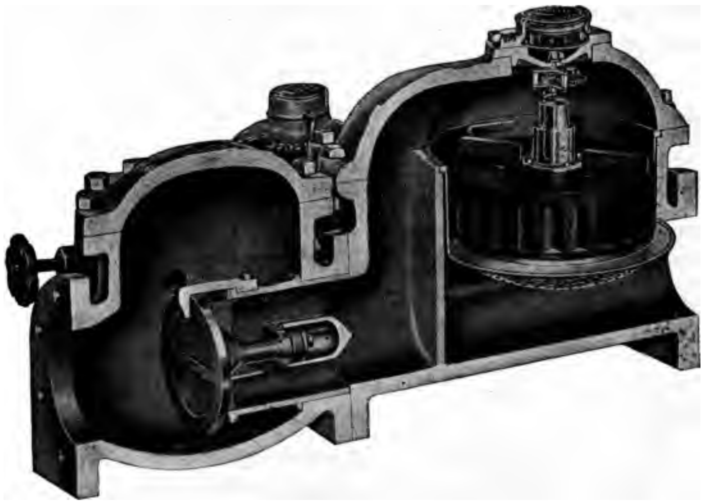


FIG. 314. — Hersey Compound Meter.

The Hersey Compound Meter * (Fig. 314) is a combination of the Hersey *torrent* and *disc* meters. The controlling-valve is of the sliding check, differential type. The by-pass is provided with suitable valves which permit its removal for repairs or test, without shutting off the main supply.

FIG. 315. — Trident Compound Meter.

The Trident Compound Meter † (Fig. 315) is a combination of the Trident crest and disc meters, which are so connected that the former measures the large flows while small flows are indicated by the latter. The change from one meter to the other is made automatically. The sum of the readings of the two meters gives the total quantity of water that has passed through the main pipe.

* The Hersey Mfg. Co., South Boston, Mass.

† Manufactured by Neptune Meter Co., New York.

The Empire Compound Meter* is a combination of the Gem and Empire meters, the latter being placed on a by-pass to measure small flows, while the former is used to measure large flows. The flow through the Gem meter is controlled by a valve of peculiar form, arranged like a pop-valve. When this valve once lifts, it is held open by a pressure much less than that required to lift it from its seat. The flow through the Gem meter is, therefore, obstructed only in a very small measure by the valve, which has a cushioned effect in seating which makes it assume its closed position without a shock.

FIG. 316. — Vertical King Disc Meter.

The Union Water Meter Co. makes a compound meter in which the disc meter is used for small streams and the Nilo meter for large flows.

540. Meters for Vertical Pipes. — A meter has sometimes to be connected with a vertical service-pipe. The Union Meter Company has modified the King disc meter, by bringing the inlet and outlet connections into vertical alignment to make it possible to connect them easily with a vertical service-pipe (Fig. 316). The mechanism in this meter remains in its normal position. These meters are made in sizes from $\frac{5}{8}$ to 1 in., inclusive. They can be placed at a convenient height for reading.

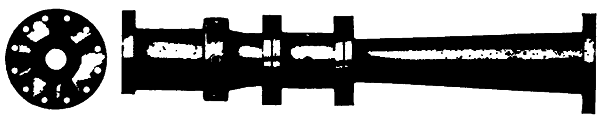


FIG. 317. — Side View.

FIG. 318. — Section, Venturi Water Meter.

541. The Venturi Water Meter† (Figs. 317 and 318‡), invented by Clemens Herschel, M. Am. Soc. C. E., in 1886, consists of two units: the meter-tube, and the indicating, recording or registering instrument. The former is essentially formed by two conica pipes, joined at their

* Manufactured by the National Meter Co., New York.

† Trans. Am. Soc. C. E., 1887, XVII, p. 228; *The Engineer*, London, August 27, 1887; *Cassier's Magazine*, March, 1899.

‡ Manufactured by Builders Iron Foundry, Providence, R I.

smaller ends by a short cylindrical pipe of the same diameter, called the throat. The conical pipes are made of any kind of convenient material, usually of cast iron, and have the same diameter at their two outer ends as the pipe, the discharge of which is to be measured. When the flow of water is always in one direction, the conical pipe at the inlet into the meter is made shorter than the one at the outlet, but when the flow has to be measured in either direction, both pipes are made alike. The throat is usually made of, or lined with, bronze or brass. An annular passage, called a pressure-chamber, is provided both at the inlet and at the throat. Four or more small openings connect this chamber with the inside of the pipe. Openings for pipe-connections to the instrument are provided at the inlet and throat pressure-chamber of the meter-tube.

FIG. 319. — Venturi Register.

The meter-tube forms part of the pipe-line the discharge of which is to be measured. Its action is based upon the principle, first observed by the Italian philosopher Venturi, about 1791, that water flowing through a converging pipe loses pressure as it gains velocity, and, conversely, that in passing through a diverging pipe it gains pressure as it loses velocity. If the joints made by the converging and diverging pipes with the throat are rounded off, so as to offer the least possible resistance to the flow of the water, the head lost from the inlet to the throat will be almost entirely recovered at the outlet, the only loss being that caused by the friction in the meter-tube.

By placing piezometer pipes at the inlet and the throat, the change of pressure of the water while passing through the meter is clearly seen. In practice, small tubes are connected to the meter, instead of piezometer pipes, at the two points mentioned above, and are led to an instrument where the quantity of water flowing through the meter is recorded by means of the difference of pressure in the tubes at the inlet and throat. This instrument (Fig. 319), called the *register*, may be placed at any convenient place within 500 ft. of the meter.

At the back of the register there are two large vertical wells, connected at the bottom by a small pipe. One of these wells is connected to the inlet pressure-chamber, and the other to the throat pressure-chamber, by two small pipes. In each well there is a heavy float, resting upon mercury, a part of which flows from one well to the other, in direct proportion to the difference in the two pressures. This causes one float to rise and the other to descend, the movement being transmitted through substantial rack and spur gearing to the *indicator dial-hand shaft.* A cam on this shaft controls the position of the pen on the chart, and, also, the amount of the movement of the counter dial figures.

The instrument is provided with three dials. *The indicator dial* shows the exact rate of flow in pounds per hour, gallons per day, etc. It is over 30 inches in circumference, and can be read within an accuracy of $1\frac{1}{2}$ per cent. It is of great value in making short tests.

The counter dial shows the total quantity of water passing through the meter, expressed in gallons, cubic feet, pounds, etc. When flow occurs through the *meter tube,* a movement of the long dial hand — which is the most rapidly moving hand — is readily perceptible. Inside of the graduations of the large dial, there are four smaller dials, which show the graduations for smaller periods. All of the dial-hands revolve in a clockwise direction, rendering the counter-totals easy to read, and the movement is continuous.

The chart recorder dial contains a large circular chart, 12 inches in diameter, which gives a continuous autographic record of the rate of flow through the meter tube. This record is completely visible at all times.

In general practice, Venturi meters are ordinarily used to measure water flowing in one direction only. When they were introduced in London, which was, at that time, supplied by eight different water companies, it became necessary to make the meters register water flowing in either direction so that the water furnished by either of two adjoining companies to the other might be ascertained. To accomplish this purpose, both conical pipes were made alike, and each was furnished with a pressure-chamber like that provided for the throat. By means of a system of cocks, the pressure-chambers of the conical pipes were so connected with the recording apparatus that either might be used or shut off.

The velocity of the water at the throat may be found by the following formulas, and by multiplying the area of the throat by the velocity the total discharge is obtained. This computation is not necessary when a register is used.

Let P = static pressure at meter in feet of water (water at rest);
P_1 = pressure at inlet in feet of water (water flowing);
P_2 = pressure at throat in feet of water (water flowing);
$H_v = P_1 - P_2$ = *the Venturi head;*
v_1 = velocity at inlet in feet per second;
v_2 = velocity at throat in feet per second.

Assuming the area of the throat to be $\frac{1}{9}$ of the area of the pipe; we have $v_2 = 9\,v_1$. The *heads* producing the velocity at the inlet and at the throat will be respectively $\frac{v_1^2}{2g}$ and $\frac{v_2^2}{2g}$.

$$P_1 = P - \frac{v_1^2}{2g};$$

$$P_2 = P - \frac{v_2^2}{2g};$$

$$P_1 - P_2 = \frac{v_2^2}{2g} - \frac{v_1^2}{2g} = \frac{80\,v_2^2}{81 \times 2g} = H_v.$$

$$\text{Velocity at throat} = \sqrt{\tfrac{81}{80}}\,\sqrt{2\,gH_v} = 1.0062\,\sqrt{2\,gH_v}. \qquad (110)$$

Experiments with Venturi meters 12, 48, and 108 ins. in diameter give $v = 0.90$ to $1.02\,\sqrt{2\,gH_v}$ for extreme values within the limits in which these meters can be used. For more than three-quarters of the extreme range of capacity of the meter, the coefficient for the three meters mentioned above is only 0.94 to 1.00. After a meter has been properly rated, it will record the flow within a small fraction of one per cent.

The loss of head due to friction in these meters may usually be kept at less than one foot of water pressure. Venturi meters installed by the East Jersey Water Co. (p. 337) in a 48-in. pipe-line lost only 4 ins. head in passing 25,000,000 gals. per day. A meter of the same size, used by the East London Water Co., passes 37,000,000 gals. per day with a loss of 1 ft. head, and 60,000,000 gals. with a loss of $2\frac{1}{2}$ ft. head.

The great advantage possessed by the Venturi tube is that it has no inner parts that might get out of order. It will pass fish and any object that can float through the throat, and may be used for sewage, hot brine, etc. The meter will measure the discharge of any pipe from $\frac{1}{4}$ in. to 20 ft. or more in diameter,* the only objection to its use in the small pipes (say 6 ins. or less in diameter) being the fact that the cost of the register is independent of the size of the pipe, and is the same for any size of pipe.

* Venturi meters 17.5 ft. in diameter have been placed in the Catskill aqueduct (p. 312).

542. *First Venturi Meters.* — The East Jersey Water Co., which supplies Newark and several other towns in New Jersey with water, measures the total quantity of water received through its two 48-in. riveted steel pipes by two 48-in. Venturi meters and the water sold to the consumers by ten similar meters, 12 to 48 ins. in diameter. The quantity registered by these ten delivery meters agrees usually within $\frac{1}{2}$ to 1 per cent of that recorded by the two receiving meters.

The Venturi meters installed by this company in 1890 were the first of this type to be put into actual use, and are still in service in good condition. Since then Venturi meters have been introduced extensively in water works in different parts of the world.

543. Venturi Meter Waste-detector (Fig. 320). — The Venturi meter has, also, been used as a waste-detector. For this purpose,

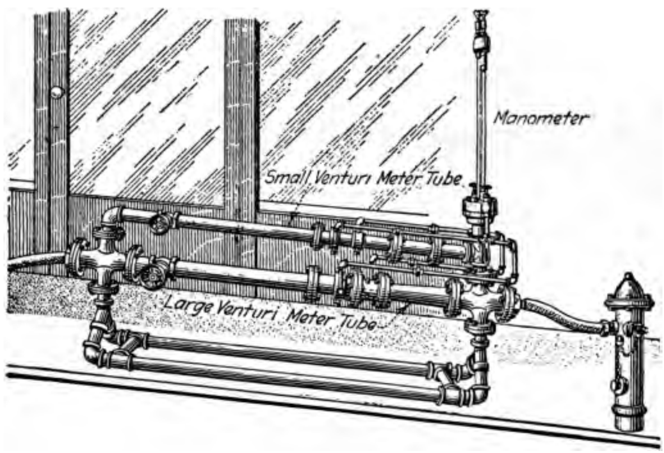


FIG. 320. — Venturi Waste-detector.

two small Venturi tubes of different, but overlapping, capacity ranges are mounted on the same stand, and connected by suitable pressure-pipes and valves to a manometer with two scales, one for each of the meter tubes. The manometer is of the barometric type. Its base is a well, connected to the inlet end of each tube, and the top interior of the glass tube communicates with the throat chamber. By means of suitable valves, either one of the Venturi tubes can be connected, at a time, with the manometer.

In using this waste-detector, the district to be investigated is cut off after midnight from the rest of the distribution system, and the waste-detector is connected by a hose to a hydrant outside of the district, and to one inside. The meters are then used to obtain the flow of water into the district. The larger Venturi tube is first used, and if the flow is too small to be measured satisfactorily with this tube, the smaller Venturi tube is used. Knowing what would be a reasonable consumption at night in the district in question, the detector will indicate the waste of water, if any exists.

The larger Venturi tube has a 3-in. inlet and a $1\frac{1}{2}$-in. throat. The inlet and throat of the smaller meter are, respectively, 2 ins. and $\frac{5}{8}$ in. The base of the detector measures $17\frac{1}{2}$ by 80 ins. The instrument measures 96 ins. over all, has a height of 20 ins., and weighs about 800 lbs. This waste-detector was first used in 1915 in St. John's, Newfoundland.*

544. The Premier Meter † (Fig. 321), invented by Arthur S. Tuttle, M. Am. Soc. C. E. in about 1901, and manufactured by the National

FIG. 321. — Premier Meter.

Meter Company of New York, is a proportional meter for use on large mains in which the discharge is determined by the actual measurement of the flow through a by-pass, this bearing a certain definite relation to that through the main. It consists of two Venturi tubes, one of which is inserted in the main and the other in the by-pass, which are joined together at their throats, the down-stream cone of the main tube being common to both tubes. The flow through the by-pass is kept within such limits as to permit of its measurement by an ordinary displacement meter, the Empire piston type meter being generally used for this purpose. By this simple arrangement, the quantity of water passing through a water main at ordinary velocities can be accurately measured. The register of the positive displacement meter is of the straight reading type, and is arranged to indicate the discharge of the main pipe. It should be enclosed in a manhole chamber large enough to permit access for inspection.

The throats of the main and auxiliary Venturi tubes are made of brass, and are encased in a cast iron chamber, to protect them from

* *Eng. News*, June 15, 1916, p. 1160; *Engineering and Contracting*, June 21, 1916, p. 555.

† *Eng. News*, May 19, 1904, p. 472; *Eng. Record*, May 21, 1904, p. 662.

injury. The remainder of the meter, with the exception of the registering apparatus, is built of cast iron. The main tube is made with flanged, hub, or spigot ends, as may be desired. The meter is equipped with a gate on each side of the registering device, to make it possible to remove and test the latter, without interrupting the flow through the main pipe. The registering apparatus is provided with a fish-screen and a hand-hole for the removal of foreign matter.

The operation of the meter is based upon the well established principle that the temporary loss of head at the throat of Venturi tubes of similar design, but of different sizes, is the same in each for any given throat velocity. It follows, therefore, that when the same temporary losses of head exist at the throats of each of two similar tubes of this character, the velocity in each main must be the same, and that the flow through the tubes must be in direct proportion to their respective areas. In order to overcome the slight friction of the recording apparatus, some special features are introduced in the meter, to make the velocity in the by-pass somewhat less than in the main pipe. This removes, also, the danger of floating bodies entering the by-pass. The accuracy of the Premier meter has been proved by many experiments* and service records show that it is maintained in long continued uses.

While the establishment of the proportional flow through the by-pass is controlled by the design of the meter and can be determined by the application of hydraulic laws, in order that this may be unquestionably established, each meter is invariably subjected to tests before leaving the factory, the tests consisting of weir measurement for the larger meters and of tank measurement for the smaller sizes.

545. Simplex Water Meter. — The Simplex Valve and Meter Company, of Philadelphia, Pa., devised in 1904, and finally perfected in 1909, a register which indicates the rate of flow, and records the quantity of discharge, of water passing through a Venturi tube, pipe, conduit, canal, or over a weir. The great advantage of this device is that it will measure with equal accuracy flows varying in velocity, from as low a rate as half a foot per second to the highest velocity.

The essential part of this device consists of a register (Fig. 322) which has suitable pipe connections with a Venturi tube, Pitot tubes, an orifice, a rectangular or V notch weir. The basic principle is a mercury vessel on which rests a float, so shaped that its vertical position is in direct ratio to the flow of water through the Venturi tube, orifice,

* Proc. Amer. Water Works Assoc., 1908, p. 247. *Eng. Record*, June 18, 1904, p. 777; *Eng. News*, June 16, 1904, p. 569.

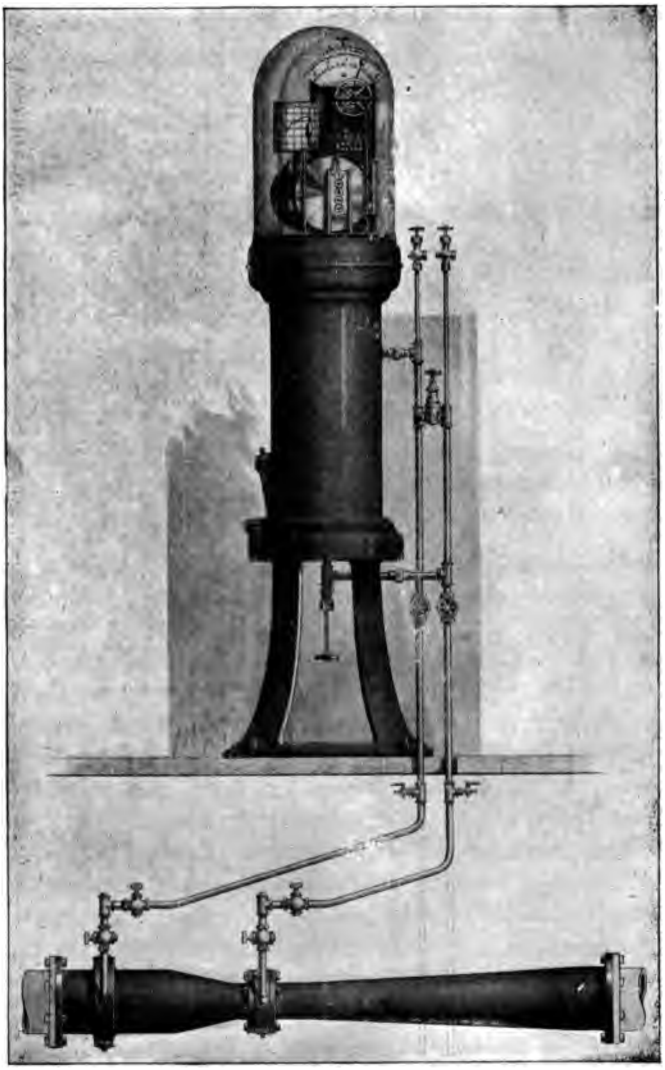


FIG. 322. — Simplex Meter.

weir, etc. The design of this float involves a beautiful principle of higher mathematics and is in every case entirely rational. In the case of the Venturi tube, orifice, or Pitot tubes, the float is hollow, and the equation of the section of its inner surface of revolution is:

$$y = \frac{a}{(x+b)^{\frac{1}{4}}}.$$

In the case of the rectangular weir:

$$y = \sqrt{\frac{a\left(\frac{1}{x^{.35}} - b\right)}{\frac{1}{x^{315}} + c}} \tag{111}$$

In the case of the triangular notch weir:

$$y = \sqrt{\frac{a\left(\frac{1}{x^{\frac{2}{5}}} - b\right)}{\frac{1}{x^{\frac{2}{5}}} + c}}. \tag{112}$$

In all of these equations,

x = the abscissa;
y = the ordinate of the section of the float;
a, b, and c are constants.

The development of these three equations constitutes as ingenious a work in applied calculus to general geometry as are those classical curves of antiquity known as the Cissoid, Conchoid and the Witch of Agnesi.

It is stated by the designer that in every case where the float was made strictly according to the curve plotted from the equation, that the actual practical results were correct within the ordinary limits of observation. In the case of the Venturi tubes and orifices, however, it was found that the flow was not proportional to the square root of the Venturi head for low velocities, and therefore the float, mathematically designed, was modified to suit these varying coefficients from a throat velocity of 4 ft. per second down to zero. Above 4 ft. per second, however, the actual velocity curve follows the mathematical curve. By means of this achievement, the necessity of cams and other limiting and friction causing devices is eliminated.

The movement of the float actuates a revolving shaft, to which is attached a hand pointing to a fixed dial with uniform graduations. A pen is attached to the shaft and moves in proportion to the angular deflections thereof. It is in contact with a rectangular chart, which is wrapped on a revolving cylinder. A traction wheel is, also, attached to the shaft, and is geared to a train of wheels operating a series of small dials, similar to those of other water meters. This traction wheel passes over the face of a revolving disc. This disc, and also the cylinder to which the chart is attached, are operated by an eight-day marine clock.

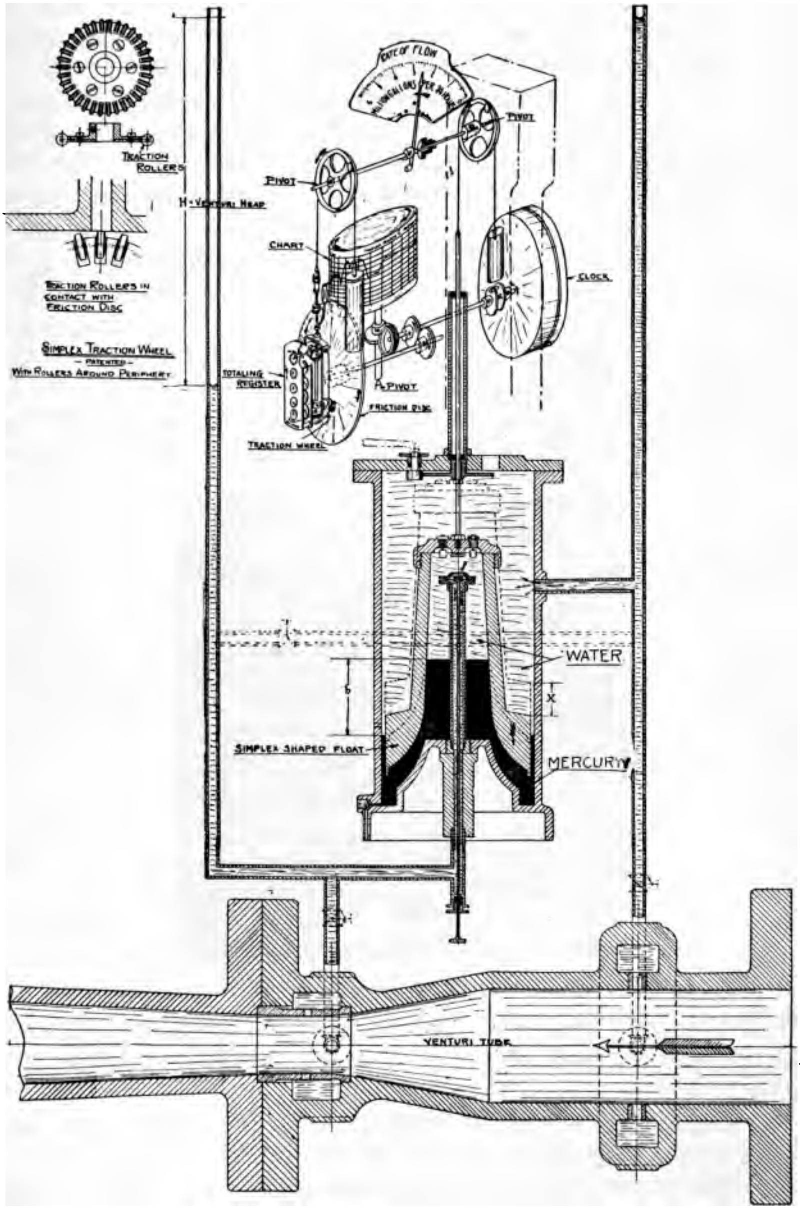


FIG. 323. — Details of Venturi Tube and Simplex Meter Register.

The entire periphery of the traction wheel is made up of a series of rollers which eliminate rubbing friction, while in contact with the face of the disc. The pen is capable of operating for months with a single charge of ink, without blotting, flooding, or failing to mark. A delicate spiral spring, whose tension is easily adjusted, keeps the pen in contact with the chart (Fig. 323), which is uniformly graduated, the abscissas representing hours and the ordinates the rate of flow. The area comprised between the datum line and the line described on the chart by the pen is in direct ratio of the total flow. It can readily be measured, and by multiplying it by the proper coefficient, the total discharge per 24 hours is obtained in gallons, cubic feet, or any other desired unit. The planimeter may, however, be set so that its reading corresponds to some multiple of ten times the unit of the chart and all calculations are thus obviated.

A glass dome, weighing less than six pounds and resting on a felt gasket, covers the rate of flow dial, chart recorder and total flow register

FIG. 324. — Battery of Water Meters.

and protects them from dust. By this arrangement the whole mechanism is at all times exposed to view. A neat metal cover is supplied when desired.

The Simplex register described above can, also, be well used in connection with a Venturi tube or with Pitot tubes.

546. The Battery System of Water Meters. — Small flows of water are measured more accurately by small water meters than by large ones. For this reason, greater accuracy is attained by using a battery of two or more small meters instead of a large meter (Fig. 324).

In 1910 such an arrangement was installed at Bay City, Mich., for measuring the discharge of an 8-in. water main by means of two 6-in. Lambert disc meters. The 8-in. main is continued for a short distance by two 6-in. branch pipes, each provided with a disc meter, and these pipes are joined again to the 8-in. main. A gate-valve is placed on each side of each meter.

The advantages claimed for this arrangement, known as the battery system, are: (1) Great accuracy by the use of small meters, which can be easily tested before leaving the factory; (2) ease in handling and setting, the bulk being divided; (3) the advantage of being able to shut off and repair either meter, without stopping the supply. For a 10-in. line, three 6-in. disc meters would be needed and for a 12-in. line, four 6-in. disc meters would be required.

The use of a number of meters, each placed in a branch pipe and provided with gates, increases the cost, and in such cases it is preferable to use either a Venturi meter or some good type of current meter.

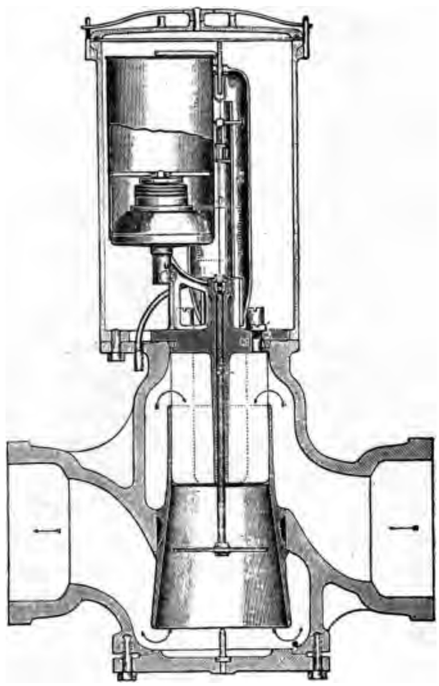


FIG. 325. — Deacon Water Meter.

European Water Meters

547. The Deacon Meter* (Fig. 325), invented about 1875 by George F. Deacon, Borough and Water Engineer of Liverpool, is placed directly in the line of a water mafn, or on a by-pass, for ascertaining the quantity of water flowing through the main. It consists of a cast iron body having either flanges or hubs at its ends, for connecting it with the water main. Inside of the meter, a conical, brass gauge-tube is placed, with its larger end at the bottom. A horizontal disc, having the diameter of the smaller end of the gauge tube, is suspended in this tube. It is attached to a counterweight, as described below, and closes the gauge-tube, when no water is passing through the meter. When water flows through the meter it forces the disc

* Manufactured by Palatine Engineering Co., Liverpool, England.

down, and passes through the annular space between the disc and the gauge-tube. As the flow increases, the disc is moved farther down by the water, and thus provides a larger opening for the water to escape.

The disc is attached to a stem which is kept vertical by a guide tube. A fine wire is fastened to the upper end of the stem. This wire passes through a practically water-tight packing-gland in the upper part of the body of the meter, and is then connected to a carriage, holding a metallic pencil, which marks the motions of the disc. To the other side of the carriage is fastened a rope which passes over a pulley and is then attached to the counterweight, made just heavy enough to draw the disc to the top of the gauge-tube, when no water is flowing. The movements of the pencil are recorded on diagram paper placed on a cylindrical drum, which is revolved by clockwork. By changing the gears, the drum can be made to revolve once in six hours, twenty-four hours or seven days, as may be desired.

The difference in pressure between the water on the two sides of the disc is just sufficient to balance the weight of the disc with the stem and the pencil carriage, and equals about six inches of water. As this difference remains constant, the flow through the meter is nearly proportional to the area of the annular opening between the disc and the gauge tube. The exact quantity of water passing through the meter for each position of the disc is determined empirically, and the diagram paper is divided accordingly and graduated to read the rate of flow in gallons per hour.

As the drum revolves, the pencil records automatically the rate of flow at a given time and the total quantity of water flowing during a given period can be obtained by averaging the rates of flow per hour and multiplying this average by the number of hours in the given period. The total quantity of water passing through the meter can, also, be obtained by means of an integrating mechanism, attached to the meter.

Deacon meters are manufactured in sizes of 3 to 36 ins. diameter, the largest size having a capacity of passing about 16,000,000 gals. per day. They have been used extensively in Great Britain and to some extent in America in connection with detection of waste of water.

548. Reciprocating Piston Meters. — *The Kennedy piston water meter* * (Fig. 326) has long been favorably known, and is used extensively in Great Britain and on the continent. It consists of a vertical cylinder of metal, in which a piston of vulcanite a is moved alternately up and down. By means of a ring of pure *Para rubber,* which rolls between the body of the piston and the internal surface

* Manufactured by the Kennedy Patent Water Meter Co., Kilmangock, Scotland.

of the cylinder, the piston is given a practically water-tight joint, which causes scarcely any friction. At each end of the cylinder, an India rubber seat *b* is placed, on which the piston forms a water-tight joint, in case it should be forced by back pressure to either end of the cylinder. This prevents undue pressure being thrown on the piston roller.

The piston-rod, after passing through a stuffing-box in the cylinder cover, is attached to a rack *c*, which gears into a pinion, fixed on the shaft. This shaft is turned, alternately, in opposite directions by the upward and the downward stroke of the piston. The rack is kept in gear and guided in a vertical line by an anti-friction roller, which is carried by a stud projecting from one of the shaft-bearing brackets.

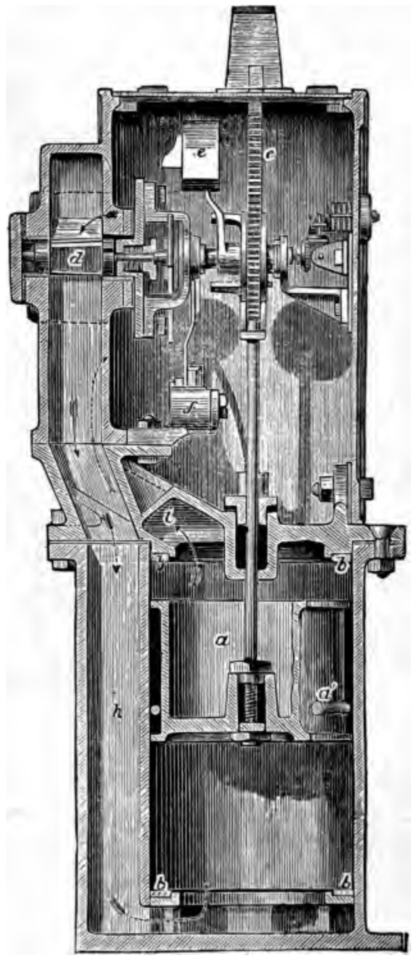


FIG. 326. — Kennedy Water Meter.

The water is directed through suitable water passages *h*, alternately, above and below the piston by the cock-key *d*, placed in the same axial line on the shaft, and is fitted with a duplex lever which is actuated by a weighed lever *e*, carried loosely on the shaft, and caused to fall, alternately, on each arm of the duplex lever. After reversing the cock, the weighted lever falls on a buffer, faced with India rubber, which, yielding before it and traveling in the same curve, gradually brings it to rest.

The reciprocating motion of the pinion shaft is converted into a circular motion in one direction by means of a ratchet, which is interposed between the pinion and the registering gear, and arranged to indicate the quantity of water that passes through the meters.

549. *The Frager piston meter* is shown in Fig. 327. It consists of two vertical cylinders G and of a top piece in which the water passages are placed, through which the water flows into each cylinder, alternately, above and below a water-tight piston K which is moved by the water pressure up and down in the cylinder. By means of the piston-rod k, the motion of each piston is transmitted to a vertical

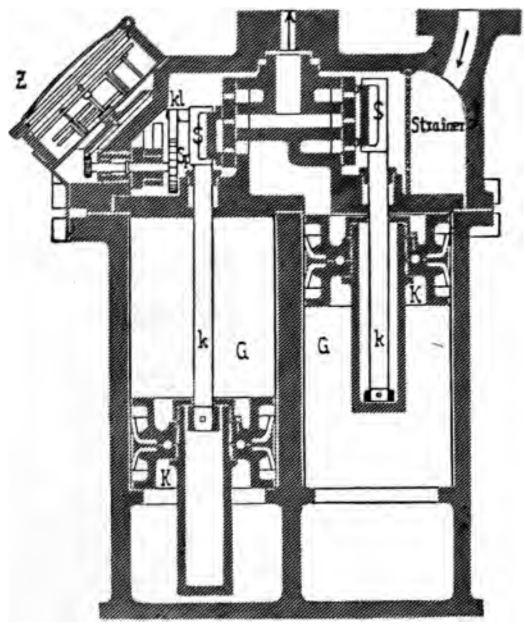


Fig. 327. — Frazer Water Meter.

slide-valve S, which regulates the flow into the adjoining cylinder. As the motion of the piston is greater than that of its slide-valve, the piston-rod is placed in a tube attached to the piston, in which it remains idle, except near the end of the stroke of the piston, when it is shoved up or drawn down by the ends of the tube, as the case may be. The reciprocating motion of one of the piston-rods is transmitted by a suitable arrangement of a ratchet and gear wheels to the train of the register L, which is placed near the top of one of the cylinders.

This type of meter is used extensively in Paris, France. In 1909 more than 30,000 of these meters were in service in that city and, also, about 45,000 of an earlier type of this meter in which the cylinders were placed horizontally.

550. *The Frost-Tavenet water meter* * (Fig. 328) has one vertical cylinder, in which a piston having a leather packing moves up and down. The meter consists of three parts. The double-acting piston *K* moves in the bottom part *U*; the middle part *M* contains the slide-valve, water passages, etc., by means of which the up and down motion of the piston is controlled; and the top part *O* forms the cover of the meter. The water is admitted, alternately, below and

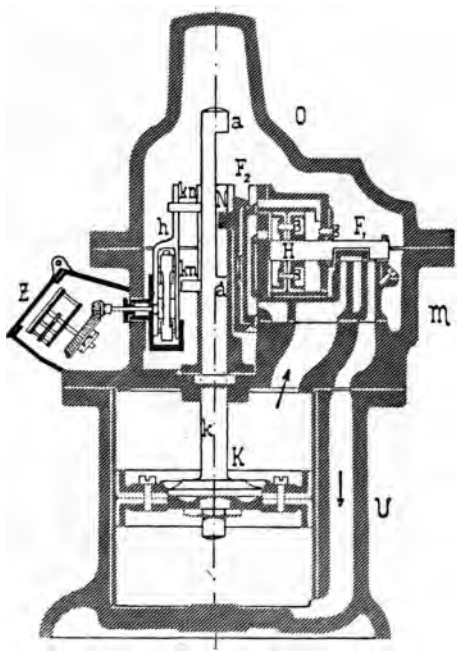


FIG. 328. — Frost-Tavenet Water Meter.

above the piston, by the shifting of the horizontal slide-valve *F*, which is actuated by a small, auxiliary piston *H*. This piston itself is moved by a vertical slide-valve having a cam *N* on its back. The slide-valve is moved up and down by the projections *aa* on the rod *K* of the main piston. Two other projections *Kn*, attached to the rod of the main piston, move the lever *h* forwards and backwards. This lever moves the pawl of a ratchet-wheel and thus changes the reciprocating motion of the piston into a circular motion which is communicated to the train of the register *L*. The inlet and outlet of the water are, respectively, on the front and back of the middle piece *M* of the meter.

* Manufactured by Michel & Company.

551. Current Meters. — *The Everett turbine water meter* (Fig. 329). The water flows through the meter in the direction indicated by the arrows and moves the cylindrical drum *C*, in whose outer surface spiral grooves are cut, as in axial turbines. On the spindle of this drum there is a worm-gear which communicates the motion through a train of gear-wheels to the register.

552. *The Michel turbine water meter* (Fig. 330) is made by the Compagnie Michel of France. After passing through a screen, the water flows through the guide wheel *b* to the *runner a* which is made of hard rubber. After leaving this wheel, the water flows out in an axial direction. The motion of the wheel is transmitted by a worm-gear to the wheel-train of the register.

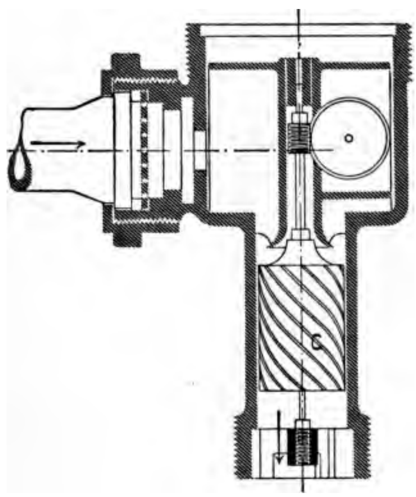


FIG. 329. — Everett Water Meter.

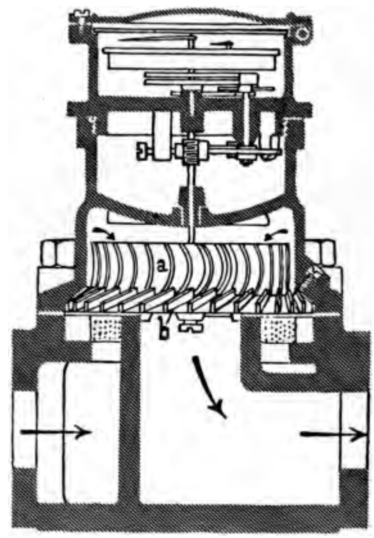


FIG. 330. — Michel Turbine Water Meter.

553. *The Andrae wing-wheel water meter* (Fig. 331) has an equal number of symmetrical tangential inlet- and outlet-ports, by means of which the water is admitted and discharged uniformly over the whole wheel. The motion of the wheel is communicated by a train of gears to the register.

The outlet-ports are placed opposite the inlet-ports, but at a higher level. The ports are bored in a housing in which the wheel is placed. They all have the same diameter, and are on the same tangential angle, but that of the outlet-ports is in the opposite direction of the angle of the inlet-ports. The symmetrical arrangement of the ports prevents all one-sided reaction of the wheel, in whatever direction the water flows. In case of back pressure, the ports change their functions, the outlet-ports becoming the inlet-ports, and *vice versa.* In this case, the register is, also, worked backwards, the quantity of water flowing from a house being thus deducted from the total quantity that had been previously recorded as being delivered.

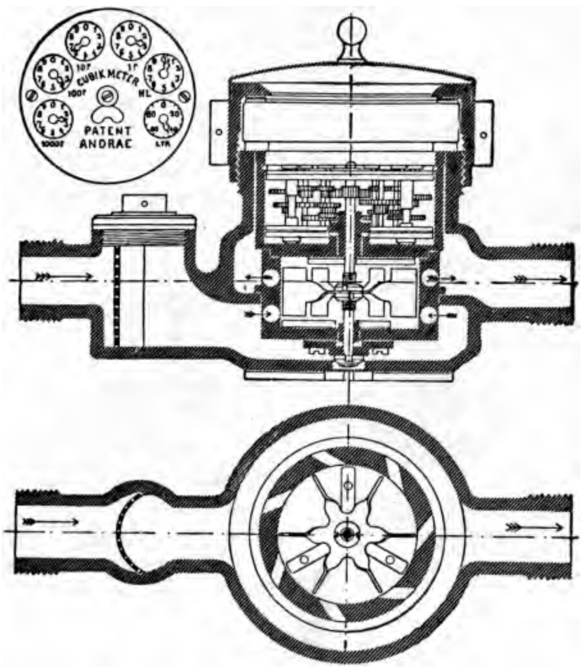


FIG. 331. — Andrae Water Meter (with Wet Register).

For small meters up to 40 mm. in diameter (about 1½ ins.), the casing is made of bronze; but for larger meters good cast iron, suitably protected against corrosion, is used. The gearing of the register is made either *wet* or *dry*, as desired.

This meter, which is the first one in which the register was arranged to work forwards or backwards, was first introduced in 1896. It is very accurate and is largely used in Germany.

554. *The Von Schinzel water meter* (Fig. 332) is manufactured in Vienna and is used extensively in Austria. It is made largely of hard rubber, which is used for the register case, the sides and bottom of the measuring chamber of the wheel, and the gear-wheels. The pinions, the spindles, and the wheels are made of delta metal.*

* This composition is made of copper and zinc, with about three per cent of iron. It shines like gold, is as hard as steel, and does not rust nor become coated with verdigris.

The covering of the measuring chamber has radiating ribs, which prevent all eddies. The meter has no regulating device. For water carrying deposits, the meter is provided with a sand-pot at the bottom.

The meter shown in Fig. 332 has a dry register, but this meter is, also, made with a wet register. In the latter case the measuring chamber is made of hard rubber, and the casing of the register of tinned bronze.

555. *Siemens & Halske Water Meters.* — The firm of Siemens & Halske, of Berlin, Prussia, has been engaged in the manufacture of water meters since 1858, and has developed a number of types of these devices. The earlier meters, made by this firm, were made with dry registers, but the later models have all wet registers.

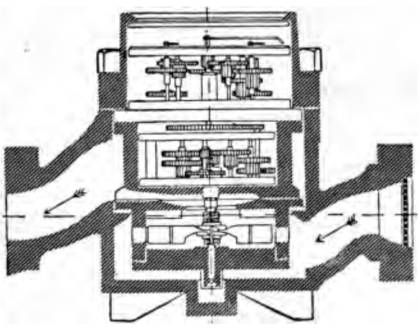


FIG. 332. — Schinzel Water Meter.

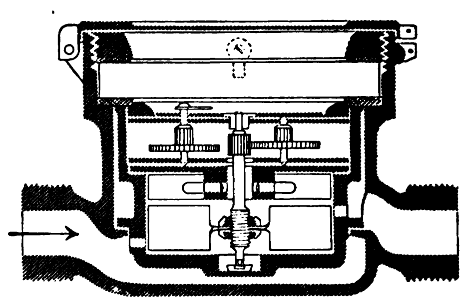


FIG. 333. — Siemens-Halske Water Meter.

Fig. 333 shows a current meter made by the above-mentioned firm, according to one of its models. The water enters at the bottom of the meter, and it passes through a horizontal strainer depositing all solid matter in a sedimentation box, without clogging the strainer. The water then flows through a number of equally distributed tangential ports into the measuring chamber, where it turns the runner, which is a wheel having straight blades.

The spindle of this runner rests on an agate bearing. Above the runner there are baffle-plates, which revolve on horizontal axes. They serve to check the rotation of the runner, when the water supply is shut off, and, also, to regulate the meter. This regulation does not reduce the capacity of the meter.

556. *Siemens and Halske's Compound Water Meters.* — In this meter large flows pass through the main pipe, and are measured by a current meter, placed in this pipe, while small flows are turned into a by-pass, which is provided with a meter capable of measuring small quantities of water. By adding the readings of the two meters, the total quantity of the water flowing through the pipe, in a given time, is obtained.

In the American compound water meters (p. 528), the change of flow from the main pipe to the by-pass, and *vice versa*, is effected by an automatic valve, which closes quickly, when the flow has been reduced to a certain quantity. If this valve does not open wide enough, at the proper time, or should open and close a number of times, when the pressures on it are in equilibrium, a certain quantity of water, too small to be measured by the main meter, might flow through the pipe, without being registered. Siemens and Halske have avoided this difficulty in their compound meter by using two automatic valves, one placed in the main pipe and the other in the by-pass. By a suitable arrangement, one of these valves is always closed, when the other is open, and thus the whole flow must either pass through the main pipe or through the by-pass. In order to make the automatic valves open and close quickly, the weight of each valve is largely reduced, as it starts to open, and correspondingly increased as it is about to close. This makes the valves very sensitive and insures a quick opening and closing.*

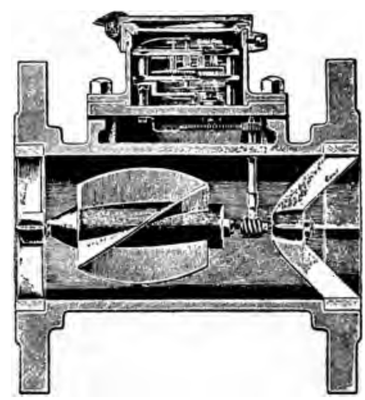


FIG. 334.—Woltmann Current Meter.

557. *Woltmann Water Meter.*—The application of the Woltmann current meter to a water meter was first made by A. Thiem, of Leipzig, Germany, in 1897, in connection with the water works of Naunhof, Germany, where large quantities of water had to be measured with a minimum loss of pressure.† This meter was manufactured by H. Meinecke, of Breslau, Prussia, now a stock company under the name of "Die Breslauer Metallgiesserei," and an extensive series of tests of different sizes of this meter were made by Herr Rother, of Leipzig, Germany.‡

Figure 334 shows a Woltmann water meter. It consists of a piece of flanged pipe, which is made of bronze for the smaller sizes of meters,

* For a full description of these valves see Otto Lueger, "Die Wasserversorgung der Städte," Band 2, S. 415, Leipzig, 1908.

† A. Thiem, "Der Woltmann Flügel als Wassermesser," Journal für Gasbeleuchtung und Wasserversorgung, 1898, S. 260.

‡ Rother, "Ueber Fortschritte in der Vervendung Woltmannscher Flügel zur Wassermessung," Journal für Gasbeleuchtung und Wasserversorgung, 1900, S. 725.

and of cast iron for the larger sizes. One of the well-known Woltmann current meters is placed in the center of this pipe, the journals of its spindle being supported by suitable frames. A worm on this spindle transmits the motion of the wheel to an intermediate gear, placed on top of the pipe, and this, in turn, moves the wheel train of the register, which is kept dry, and is separated from the intermediate gear by proper packing. The wheel itself, with the exception of its metal spindle, is made of celluloid and is hollow. Its specific gravity is practically the same as that of water, and the shape of the wheel is such that the number of revolutions it makes corresponds almost exactly to the quantity of water flowing through the pipe. This relation is given mathematically by the formula:

$$Q = h \times a \times r = \text{cubic feet per second,} \qquad (113)$$

where

h = pitch of screw;
a = area of pipe in square feet, less the area of a cross-section of the wheel;
r = revolutions per second.

One of the advantages of this type of water meter is its extreme sensitiveness, which enables it to record very small flows of water. Its superiority, in this respect, over an ordinary current meter is shown by the following comparison of these meters given by Lueger, in "Die Wasserversorgung Der Städte," Band 2, S. 417, Leipzig.

COMPARISON OF WATER METERS

Type of water meter	Diameter in millimeters	Capacity with loss of 10 meters in head. Cubic meters per hour	Limit of accuracy registry = 2% Liters per second
Woltmann*	100	500–600	0.75
Ordinary current meter	250	500–600	1.7–1.9

* Known also as the Meinecke meter.

From the above table it appears that a Woltmann water meter of 100 mm. (3.67 ins.) diameter passed the same quantity of water as an ordinary current meter of 250 mm. (9.14 ins.) and showed much greater accuracy in registering both small and large flows. The former meter is much cheaper than the latter. These advantages are also found in the larger sizes of the Woltmann water meter.

Straineers and fish-traps or sediment boxes are not generally needed for these meters, as any ordinary amount of sediment, etc., can pass through the meter without causing damage. When these meters are used for measuring water carrying a large quantity of foreign matter — such as that coming from newly driven wells, etc. — the work attached to the main spindle of the meter, and the gear-wheel it drives, are protected by a proper box.

The Woltmann meter cannot be used to advantage for smaller diameters than 50 mm. (1.83 ins.). It is manufactured in various sizes from 50 to 500 mm. (1.83 to 18.3 ins.) in diameter, the weights of these two extreme sizes being respectively 7.5 and 200 kilograms (16.53 to 441 lbs.).

This meter is well adapted for measuring large quantities of water going to different districts of a water supply system, to large factories, etc., and, also, for testing the capacity of pumps. It can be made either of the same diameters of the main in which it is to be placed, or it can be given a smaller diameter, and form the throat of a Venturi meter. With this arrangement there should be a straight piece of pipe, about 1.5 to 3.5 ft. long, back of the meter, and a similar

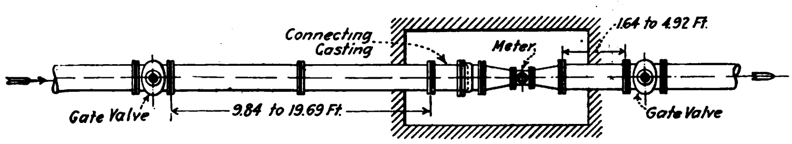


FIG. 335.

piece of pipe, about 10 to 20 ft. long, in front of the meter, in order to insure as regular a flow at the meter as possible.

Experiments have shown that this type of the meter can be placed, also, in a vertical pipe-line, without impairing appreciably its accuracy.

By means of an electrical contact, made in its register, the Woltmann meter can be made to record, on graduated paper, mounted on a drum, the variations in the quantity of water flowing through a water main, and the total quantity per day. The drum is made by clockwork to revolve uniformly once in twenty-four hours. The horizontal lines on the paper denote the rates of flow, while the vertical lines mark the different hours.

For larger mains, having a diameter of more than 8 ins., the flow may be measured accurately by passing the water through two smaller pipes, each provided with a meter. Further on these pipes are joined again (Fig. 335). By this arrangement, a greater sensitiveness for small flows can be obtained than would be secured by using one large meter. Gate-valves should be placed on each side of the meters, and, also, a fish-trap (sediment box) on the up-stream side.

558. Meter Boxes. — Water meters are usually placed in the basements of buildings, but occasionally they are set outside of a building, and protected by a suitable box. There are some advantages in adopting the latter plan. It makes the meters always accessible to the officials of the water works, and prevents tampering with them. It also protects the meters from injury when fires occur. Meters placed in ordinary cellars are apt to freeze in very cold weather. This does not occur when the meter is placed in a good box outside of the building.

FIG. 336. — Ford Meter Box.

The Ford Meter Box Company, of Wabash, Ind., manufacture the meter box, shown in Fig. 336, which is provided in cold latitudes with a double cover. It is known as the Wabash box. The meter is attached to the water pipe without any threaded fittings, the joint between the meter spuds and the inlet and outlet fittings being made by compression. A patented yoke serves to hold the meter securely in position, and can be removed in one minute's time, without the use of tools. This box protects the meter pipes and fittings securely against freezing, fire, and tampering. The lid can be removed by turning a special bolt provided with a pentagonal head.

The Clark Meter Box, manufactured by the H. W. Clark Company of Mattoo, Ill., is shown in Fig. 337. On account of its roomy construction, all sorts of necessary appliances can be installed in this box, and the large volume of air about the meter gives perfect protection against frost. This object is also assisted by the small area of cover exposed to the air. These boxes are made in over a hundred different types, to fit all climatic conditions.

*The Jersey Meter Box** (Fig. 338) consists of a cast iron top which is set on a concrete or tile housing in which the meter is placed. It is provided with a locking device consisting of a lever which engages beneath a horizontal lug. When released, it rides up on an

* Manufactured by the S. E. T. Valve and Hydrant Co., New York.

inclined plane or lug which helps to push out the cover against frost resistance.

559. Testing Water Meters. — In selecting water meters, the different types of meters should be compared as regards sensitiveness,

Fig. 337. — Clark Meter Box.

accuracy of registration, loss of pressure between the inlet and outlet, and endurance. The simplicity of construction and facility of making repairs should, also, be considered.

In testing fourteen different types of meters, J. W. Hill, M. Am. Soc. C. E., found that several of the $\frac{5}{8}$-in. meters would register a flow of 10 to 12 gals. per hour with less than 10 per cent error.* For a flow of 10 gals. per minute the loss of pressure was 6 to 8 lbs. per square inch in disc meters, and 7 to 13 lbs. in piston meters. For a flow of

* Trans. Am. Soc. C. E., 1899, XLI, p. 326.

5 gals. per minute these losses of pressure were reduced respectively to 2 to 2½ and 2 to 3 lbs.

J. Waldo Smith, M. Am. Soc. C. E., obtained, in testing seven ⅝-in. meters, a registration of 95 per cent of the flow with rates of 3 to 24 gals. per hour.* These experiments showed, also, that after delivering quantities of water corresponding to a service of 35 to 45 years, the best meters had lost little of their sensitiveness or accuracy of registration. The loss of pressure in the ⅝-in. meters tested by Mr. Smith varied from 3 to 12 lbs. per square inch for a rate of flow of 10 gals. per minute. For a flow of 5 gals. per minute the loss of head varied from 1 to 3 lbs.

FIG. 338. — The Jersey Meter Box.

Otto Poetsch tested in 1911 in Milwaukee, Wis., a large number of house meters, averaging about ¾ in. in size.† He found the average slip for 3431 piston meters to be 3.15 per cent, and the average for 1955 disc meters to be 0.6 per cent. The piston meters had been in service from 15 to 23 years, prior to the tests being made, while the disc meter had been in use for about 10 years.

Tests made by J. A. Cole in Des Moines, Iowa, on 1064 meters, which had been in service for 1 to 15 years, showed a loss of only 1½ per cent in accuracy of registration.‡

In tests of water meters made in East Orange, N. J., in 1908 and 1909, two ⅝-in. Trident disc meters and two ⅝-in. Empire meters ran constantly under full pressure for 10½ months, without failure or loss in accuracy of registration. The quantities of water discharged by the meters were equivalen to what would be used by a family of 5 persons in about 95 years.§

All water meters should be carefully tested before being installed, and a card index should be kept giving the history of each meter, and the results of tests made. After being in service for a certain number of years, or after having passed a quantity of water exceeding the average consumption equivalent to such a number of years' service, a meter should be retested. The minimum frequency with

* Trans. Am. Soc. C. E., 1899, XLI, p. 359.

† *Eng. Record*, Jan. 13, 1912, p. 34.

‡ Technology Quarterly, June, 1907.

§ Report of Board of Water Commissioners of East Orange, N. J., 1909.

which such periodical tests should be made, as regards either the years of service, or the quantities of water discharged, is given in the following table:

TABLE LXVIII. — PERIODICAL TESTS OF WATER METERS

Size of meter in ins.	Water discharged or years of service before a meter should be retested	
	Water discharged in cu. ft.	Years of service
$\frac{1}{2}$	200,000	6
$\frac{5}{8}$	200,000	6
$\frac{3}{4}$	300,000	5
1	400,000	4
$1\frac{1}{2}$	1,000,000	3
2	1,000,000	3
3	2,000,000	2
4	3,000,000	1
6 or larger	5,000,000	1

In testing meters, care must be taken to distinguish errors due to faulty testing apparatus from those due to the meter itself. For volumetric tests, the tank that receives the water discharged by the meter should be accurately calibrated. If the water in the tank is to be weighed, the tester must make sure that the scales are correct and sensitive within a pound. In weighing one cubic foot of water, an error of one-half pound would cause an error of nearly 1 per cent in the results.

If possible, enough water should be discharged through the meter to make the hand of the test dial make one or more complete revolutions, so as to eliminate inaccuracies in the subdivisions of the dial, which sometimes exist, and may introduce an error as high as 20 per cent in the registration.

The temperature of the water must be considered in testing hot water meters, but may be neglected when only cold water is passed through the meter.

If a meter does not register correctly, the error may be due to two causes: faulty construction of the meter, or improper gear reduction. In all types of positive displacement meters, there is a certain amount of piston clearance. Although this clearance is usually only about 0.002 in., it will permit, even in small size meters, a rate of flow of about 100 gals. per day to pass through the meter without moving the piston. If the flow of water is just sufficient to move the piston, about 80 per cent of the flow may leak past the piston. As the rate of flow increases, the leakage becomes less until a flow is reached at which the leakage is a minimum. If the flow becomes still greater, the leakage will increase slightly again, until the capacity of the meter is reached.

On account of this variable piston leakage for different rates of flow, it is evidently impossible to put in a meter spur-gearing which will transmit to the register gear-train correct indications for all rates of flow. The spur-gearing must, therefore, be selected to give the desired accuracy of registration at average rates of flow.

Fig. 339 shows an average accuracy curve for new $\frac{5}{8}$-in. disc meters. According to this curve, meters of this size and type will register

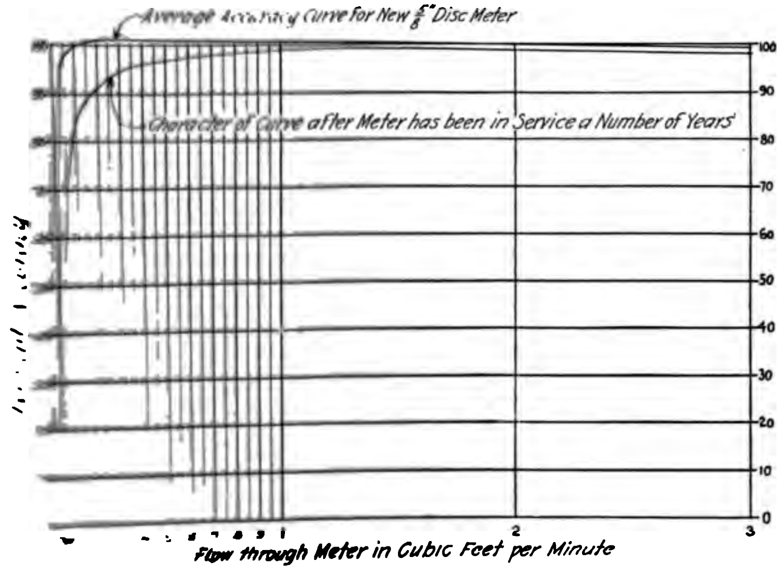


Fig. 339.—Average Accuracy Curves for a New $\frac{5}{8}$-inch Disc Meter.

[illegible] of the total amount of water passing through [illegible] from about 0.07–2.5 cu. ft. per minute. [illegible] years service, the registration will increase, in most [illegible] per cent, as fine particles of suspended matter [illegible] upon the disc and disc chamber, thus reducing [illegible] clearance and causing the piston to make a [illegible] revolutions per unit quantity discharged. After [illegible] ordinary service, wear of the moving members [illegible] effect upon the accuracy of the registration, [illegible] by the lower curve in Fig. 339.

[illegible] to pass the desired test, the cause may be [illegible] spur-gearing, and by changing the spur reduction, the meter may be made to pass the required test. For this reason a water company should always keep on hand a number of changes of spur-gearing for the different types of meters which it installs.

The loss of pressure varies greatly in different makes of meters. At flows equal to their nominal capacities, $\frac{5}{8}$-in. disc meters, made by different manufacturers, lose about 8 to 30 lbs. pressure per square inch. In general, the pressure loss in any meter should not be much greater than 15 lbs. per square inch with a flow corresponding to the nominal capacity of the meter.

The practical tests required in different water works as regards accuracy of registration vary somewhat. In the following table we give the standard tests for water meters required by the Department of Water Works, Gas and Electricity of the city of New York.

TABLE LXIX. — TESTS OF WATER METERS REQUIRED BY THE CITY OF NEW YORK

Diam. of meters in ins.	Diam. of orifice in ins.	Water used in test, cu. ft.	Per cent registration required at the meter testing station						Tested on the premises. All types of meters
			Positive meters		Current meters		Hot water meter		
			New	Used or repaired	New	Used or repaired	New	Used or repaired	
$\frac{5}{8}$	$\frac{1}{16}$	1	98–102	95	.....	.....	.....	.....	.....
	$\frac{1}{8}$	1	.....	.....	.....	.....	85	85	80
	$\frac{5}{8}$	10	98–102	98–102	.....	.....	98–102	98–102	95–105
$\frac{3}{4}$	$\frac{1}{16}$	1	98–102	95	.....	.....	.....	.....	.....
	$\frac{1}{8}$	1	.....	.....	.....	.....	85	85	.....
	$\frac{1}{4}$	1	.....	.....	.....	.....	.....	.....	80
	$\frac{3}{4}$	10	98–102	98–102	.....	.....	98–102	98–102	95–105
1	$\frac{1}{8}$	1	98–102	95	.....	.....	.....	.....	.....
	$\frac{1}{4}$	1	.....	.....	.....	.....	85	85	80
	1	10	98–102	98–102	.....	.....	98–102	98–102	95–105
$1\frac{1}{2}$	$\frac{1}{8}$	1	98–102	95	.....	.....	.....	.....	.....
	$\frac{1}{4}$	1	.....	.....	98–102	95–105	.....	.....	.....
	$\frac{1}{2}$	10	.....	.....	.....	.....	.....	.....	95–105
1	.....	.....	.....	.....	.....	.....	.....	.....	95–105
	$1\frac{1}{2}$	10	98–102	98–102	98–102	98–102	.....	.....	95–105
2	$\frac{1}{8}$	1	98–102	95	.....	.....	.....	.....	.....
	$\frac{1}{4}$	1	.....	.....	98–102	95–105	.....	.....	.....
	$\frac{1}{2}$	.....	.....	.....	.....	.....	.....	.....	95–105
	2	10	98–102	98–102	98–102	98–102	.....	.....	95–105
3	$\frac{1}{4}$	1	98–102	95	.....	.....	.....	.....	.....
	$\frac{1}{2}$	10	.....	.....	98–102	95–105	.....	.....	95–105
	1	10	98–102	98–102	98–102	98–102	.....	.....	95–105
	3	100	98–102	98–102	98–102	98–102	.....	.....	95–105
4	$\frac{1}{2}$	10	98–102	95–105	98–102	95–105	.....	.....	95–105
	2	100	98–102	98–102	98–102	98–102	.....	.....	95–105
	4	100	98–102	98–102	98–102	98–102	.....	.....	95–105
6	$\frac{1}{2}$	10	98–102	95–105	98–102	95–105	.....	.....	95–105
	2	100	98–102	98–102	98–102	98–102	.....	.....	95–105
	6	100	98–102	98–102	98–102	98–102	.....	.....	95–105

NOTE. — The water pressure at the different meter testing stations varies from about 35 to 40 lbs. per square inch.

In the city of Cleveland, Ohio, where great attention has been given to the subject of water meters, the following accuracy of registration is required:

TABLE LXX. — TESTS OF WATER METERS REQUIRED IN CLEVELAND, OHIO

Size of meter, ins.	Diameter of orifice	Rate of flow approx., cu. ft. per minute	Per cent registration required
$\frac{5}{8}$	$\frac{5}{8}$	3	98–101
	$\frac{1}{16}$	0.111	98
	$\frac{1}{32}$	0.042	90
1	1	9	98–101
	$\frac{1}{4}$ or over	1.25 or over	99
	$\frac{1}{16}$	0.111	98
	$\frac{1}{32}$	0.042	85
$1\frac{1}{2}$ and 2	$1\frac{1}{2}$ and 2	14 and 25	98–101
	$\frac{1}{4}$ or over	1.25 or over	99
	$\frac{1}{8}$	0.312	98
	$\frac{1}{16}$	0.111	97
3	3	40	98–101
	$\frac{1}{4}$ or over	1.25 or over	99
	$\frac{1}{8}$	0.312	97
	$\frac{1}{16}$	0.111	09
4	4	80	98–101
	2 or over	....	99
	$\frac{1}{4}$	1.25	95
	$\frac{1}{8}$	0.312	80
6	6	1.40	98–101
	2 or over	....	99
	$\frac{1}{4}$	1.25	95
	$\frac{1}{8}$	0.312	70

In Rochester, N. Y., $\frac{5}{8}$-in. rate meters are required to register between 99 and 101 per cent of the quantities discharged, when working under a static pressure of 50 lbs. per square inch, both for full discharge and a flow through a $\frac{1}{8}$-in. orifice.

For a flow of 0.02 cu. ft. per minute, the meters must register within 98 to 102 per cent of the correct discharge.

Special machines are made for testing water meters. Some of these *meter testers* are described below.

Meters can also be tested on the premises by means of a test meter which is connected to the meter whose accuracy of registration is to be ascertained. In such tests the same quantity of water is passed through the two meters and the quantities of flow indicated by their registers are compared. An apparatus for testing meters in this manner is described on page 566.

560. A water meter tester, manufactured by the H. Mueller Manufacturing Company, of Decatur, Ill., is shown in Fig. 340. It consists of an iron base, suitably mounted. The meter to be tested is placed on the adjustable meter-rest with the outlet toward the tank. Reducing bushings are provided so that any size of meter up to two inches

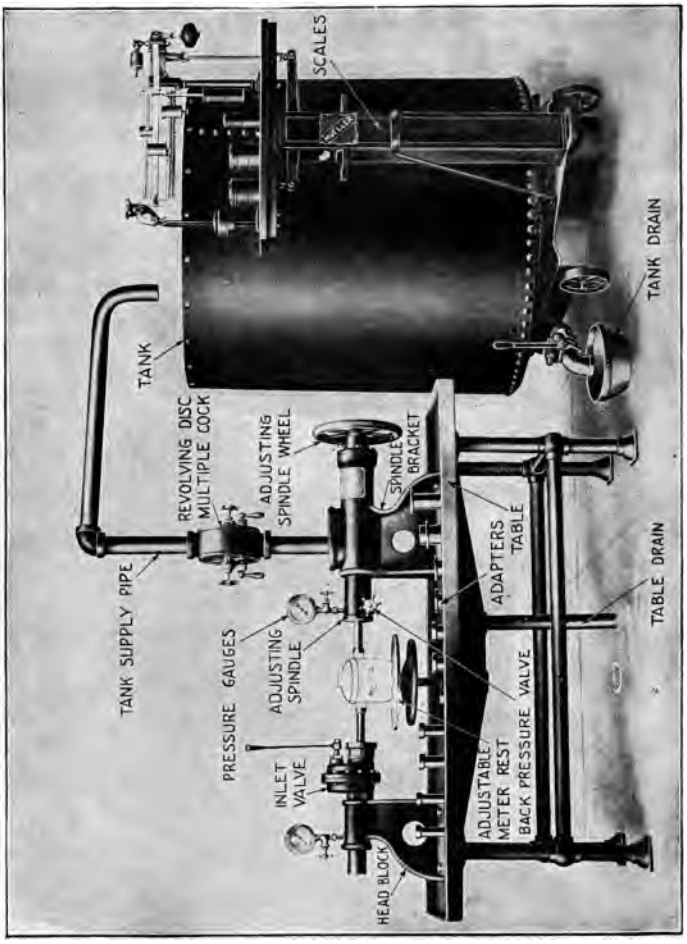


FIG. 340.—Mueller Water Meter Tester (for meters 2 inches or less in size).

can be accommodated. On the table is also mounted the head-block which brings the inlet-pipe into position and the flow is controlled by the inlet-valve. Upon the table is also mounted the spindle-bracket provided with the spindle-wheel, which adjusts the spindle to the meter. The outlet-pipe is taken from the spindle-bracket and discharges into the tank. In order that the meter may be tested conven-

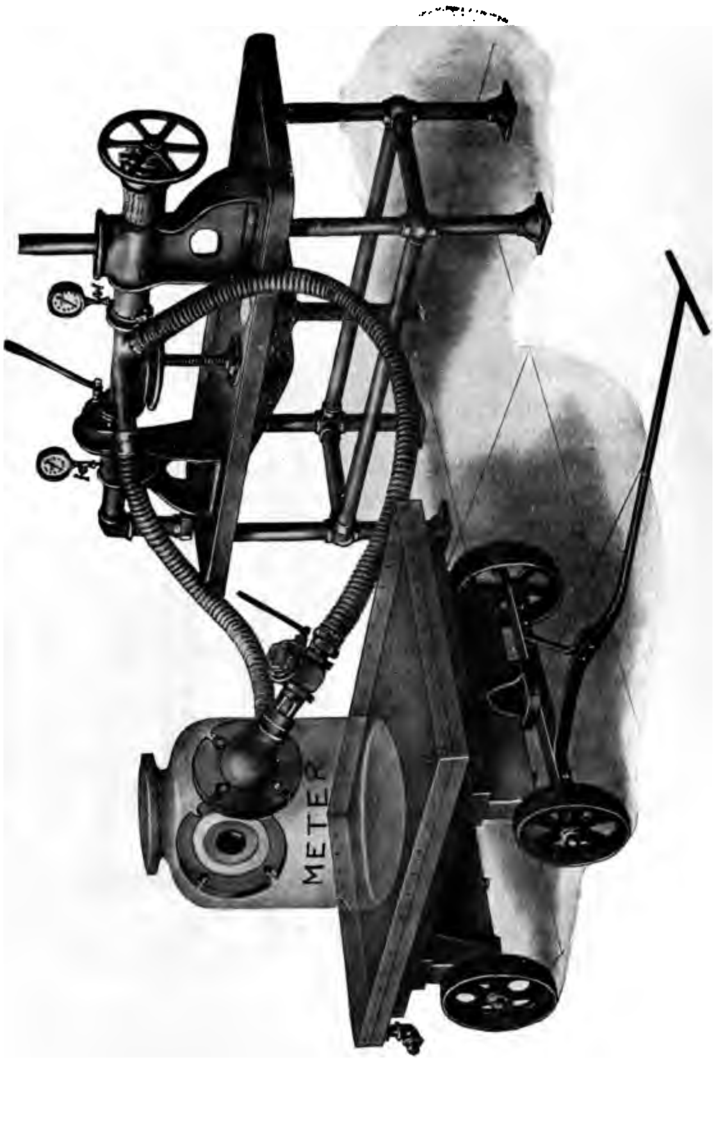


FIG. 341. — Mueller Water Meter Tester (arrangement for testing meters more than 2 inches in size).

iently with various flows, the outlet-pipe is provided with a revolving disk multiple-cock, which controls the size of the stream passing through the meter and may be regulated from one-thirty-second of an inch to two inches in size. The receiving tank is provided with an outlet, so that, when the test is completed, the water may be readily drained. This receiving tank rests on scales which are of standard design, equipped ordinarily for this work with a special double-bar

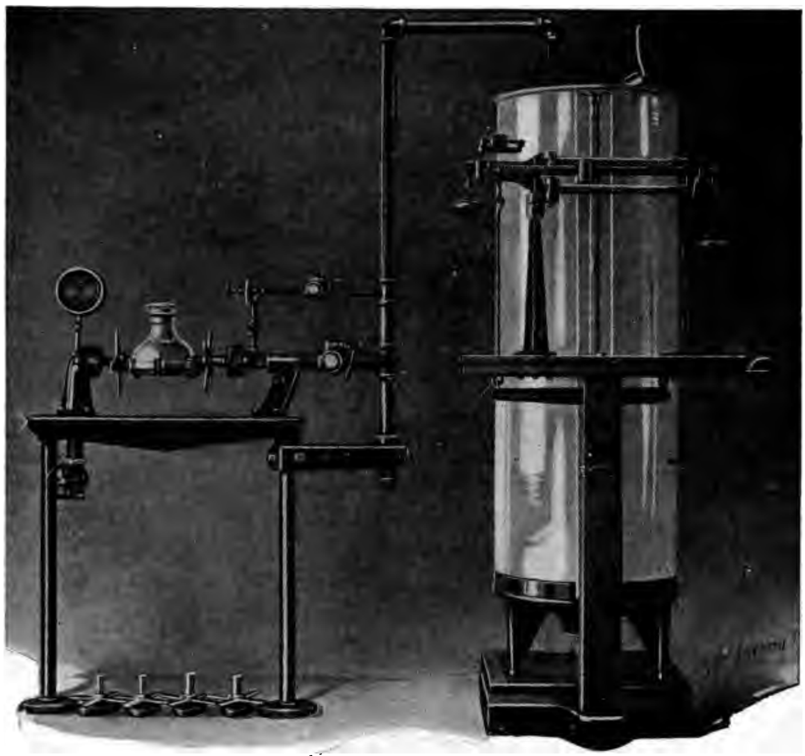


FIG. 342. — Trident Meter Tester.

scale-beam, graduated in pounds, cubic feet, and gallons, and with an auxiliary per cent bar which gives the percentage error in the meter readings without any calculations.

Figure 341 shows the method of testing meters which are more than two inches in diameter with this same size of tester. In this case the meter is mounted upon a truck as shown, and the branch connection is placed in the meter tester in position the same as the meter would be placed. With this arrangement meters as large as twelve inches can be accurately tested.

561. The Trident Meter Tester* consists of a test-bench for supporting one to six small meters in series, and of a calibrated tank, provided with a graduated gauge, in which the water is measured by volume. The bench is joined to a water main and to the tank, which has usually a capacity of one or ten cubic feet, by suitable connections. Quick-acting wedge-and-lever stop-gates placed on the inlet- and the outlet-pipe control the flow, and are used in starting and stopping the test.

The rate of flow is regulated by means of different size orifices, varying from $\frac{1}{32}$ in. diameter to the full size of the service-pipe, which are placed in the discharge pipes ahead of the quick-acting valve.

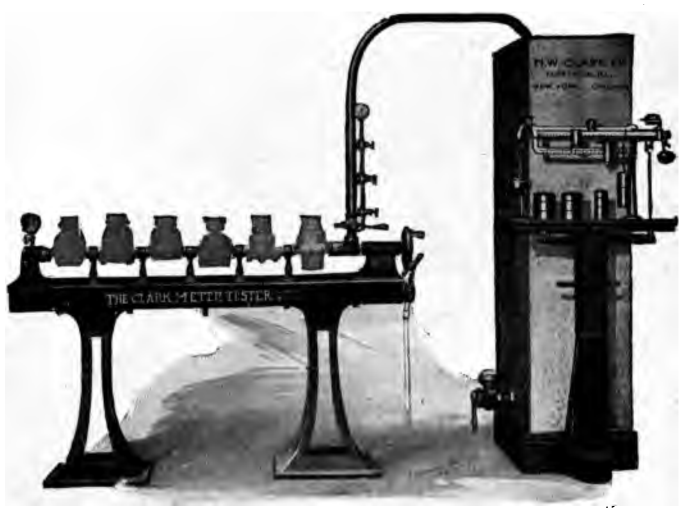


FIG. 343. — Clark Meter Tester.

For testing only one meter at a time, the tester is made as shown in Fig. 342. Two sizes of this tester are made, one for meters $\frac{5}{8}$ to 1 in. in diameter, and the other for meters having diameters of $\frac{5}{8}$ to 2 ins. With these testers the tank is placed on a sensitive scale, and the discharge is measured by weight. The scale is fitted with a special percentage-beam for testing water meters, the per cent of error being indicated directly on this beam, without any computation.

562. The Clark Meter Tester † (Fig. 343) is a solid, efficient machine, which is made in the following sizes:

* Manufactured by the Neptune Meter Co., New York.

† Manufactured by the H. W. Clark Co., Mattoon, Ill.

TABLE LXXI. CLARK METER TESTERS

Size number	Capacity, number and size of meters	Clark special test valves, number and size of openings	Water used in tests, cubic feet	Capacity of tank in cubic feet
0	One, $\frac{5}{8}''$, $\frac{3}{4}''$ or $1''$	Three, $\frac{1}{32}''$, $\frac{1}{16}''$, $\frac{1}{8}''$	1	3.75
1	One, $\frac{5}{8}''$, $\frac{3}{4}''$ or $1''$	Three, $\frac{1}{32}''$, $\frac{1}{16}''$, $\frac{1}{8}''$	1	3.75
2	Three, $\frac{5}{8}''$ Two, $\frac{3}{4}''$ or $1''$	Three, $\frac{1}{32}''$, $\frac{1}{16}''$, $\frac{1}{8}''$	10	15.75
3	Three, $\frac{5}{8}''$ Two, $\frac{3}{4}''$ or $1''$ One, $1\frac{1}{2}''$ or $2''$	Four, $\frac{1}{32}''$, $\frac{1}{16}''$, $\frac{1}{8}''$, $\frac{1}{4}''$	10	15.75
4	Six, $\frac{5}{8}''$ Five, $\frac{3}{4}''$ Four, $1''$	Three, $\frac{1}{32}''$, $\frac{5}{16}''$, $\frac{1}{8}''$	10	15.75
5	Six, $\frac{5}{8}''$ Five, $\frac{3}{4}''$ Four, $1''$ Two, $1\frac{1}{2}''$ or $2''$	Four, $\frac{1}{32}''$, $\frac{1}{16}''$, $\frac{1}{8}''$, $\frac{1}{4}''$	10	.15.75
6	One, $3''$, $4''$, $6''$, or $8''$	Four, $\frac{1}{4}''$, $\frac{3}{8}''$, $\frac{1}{2}''$, $\frac{3}{4}''$	100	160.00

The testers are provided with slow opening inlet-valves and with quick opening outlet-valves. The outlet-valve is of the *Clark fractional flow type*, fitted with indicating quadrant, which facilitates testing meters for all *fractional flows*.

Each machine is provided with *Clark Special Non-clogging Sensibility Test Cocks* and has gauges to show the initial pressure and frictional loss.

The scale has three beams graduated in cubic feet, gallons and pounds avoirdupois, with separate percentage-beam graduated for 24 per cent each way.

563. The Ford Meter Tester* is made in different sizes. The No. 1 tester is an inexpensive machine in which only one meter can be tested at a time. Fig. 344 shows a larger machine with a Fairbanks special meter testing scale. For larger cities a special testing machine is made.

563a. The National Meter Company, of New York, manufactures a very compact and convenient apparatus for testing water meters (Fig. 344*a*). It is arranged as a stand, with clamping devices to hold from $\frac{1}{2}$ to 1 in. meters, the clamping of the meter being accomplished with a hand-operated nut which clamps a bronze screw, containing the outlet-pipe of the meter. To this outlet is attached a pipe with a full-size 1 in. outlet-cock and with additional outlets in sizes of $\frac{1}{4}$ in., $\frac{1}{8}$ in., $\frac{1}{16}$ in. and $\frac{1}{32}$ in. This pipe discharges into a tank of approximately twelve gallons capacity, which is arranged on the

* Manufactured by the Ford Meter Box Co., Wabash, Ind.

Fig. 344. — The Ford Meter Tester.

Fig. 344a. — Testing Apparatus of National Meter Company.

FIG. 344*b*. — Portable Meter Testing Apparatus (Neptune Meter Co.)

platform of a sensitive scale by means of trunnions which allow the tank to be dumped into an enlarged funnel or hopper that is incorporated in the stand on which the meter is set. The funnel or hopper is attached to a sewer connection for disposing of the surplus water. The scale is arranged with a percentage-beam, and is provided with weights for either gallons or cubic feet, as may be desired.

This testing plant is particularly adapted for water works of average size. Larger meters can be tested by means of a smaller test meter which has been carefully calibrated on the testing apparatus.

563b. Testing Meters on the Premises. — Instead of testing a meter in a special machine, like those described above, it may be tested on the premises by comparing its discharges for different size openings with those of an accurate test meter. This method saves much expense and inconvenience.

The Neptune Meter Company makes a portable testing apparatus (Fig. 344*b*) for testing meters on the premises. In using it, the outlet-spud of the house meter is connected by means of a hose with a *test meter* of the Trident disc type, whose accuracy has been determined in a testing machine. The outlet of the test meter is connected with a piece of hose of sufficient length to discharge the flow into a sewer, gutter, etc. A quick-acting valve is attached to the outlet-spud of the test meter. The outlet side of the test meter is provided with a short goose-neck, near the end of which is placed an orifice of the size required for the test. The flow is controlled by the quick-acting valve.

For large size meters which are to be tested on the premises, it is convenient to place a test tee followed by a stop-cock in the service-pipe on the outlet side of the house meter, in order to avoid disconnecting the meter when it is to be tested.

CHAPTER XXXII

RECORDING INSTRUMENTS

564. Various devices have been invented for recording the height of the water surface in a stand-pipe, tank, or reservoir. The simplest contrivance which may be used for a stand-pipe or tank is a float attached to a rope which passes over a pulley and has, at its free end, a pointer for indicating the position of the water surface. This simple apparatus has been used extensively for water tanks of railroads, but occasionally gets out of order.

Some of the improved devices which have been introduced of late are described in the following pages.

565. Bristol's recording water level gauges are used extensively for recording automatically and continuously the depth or levels of water in rivers, reservoirs, water towers, etc., and, also, for recording the rate of flow of water over weirs. These gauges may be used when the recording station is above or below the water surface, whose fluctuations in level are to be charted.

The complete outfit (Fig. 345) consists of a bulb-casting of bronze, having some perforations on the lower side, a suitable length of flexible connecting tube, and a recording instrument. A flexible diaphragm of thin rubber convex downward is placed between the flanges of the upper and lower parts of the bronze casting, and is firmly held, enclosing the proper amount of air for the operation of the instrument. The casting is supported by a special bracket and chain. The flexible tube is furnished with or without steel, lead, or copper armor.

When the bulb casting has been properly installed so that its outer diaphragm is only exposed to atmospheric pressure on the inside and outside, when the water is at its lowest level, a rise of the water level will cause a corresponding increase of pressure on the bulb diaphragm, which increase of pressure will be transmitted through the flexible connecting tube to the pressure tube of the recording instrument, and will thus cause the recording pen arm to be deflected on the surface of a chart. The variations in the elevation of the surface of the water are thus transmitted directly to the pen-arm, without the intervention of multiplying devices, gears, links, levers, or other complicated mechanism. The Bristol gauges are independent of freezing temperatures as they act through air columns.

A great variety of forms of helical and diaphragm pressure tubes are used with these gauges. They have been developed to suit

FIG. 345. — Bristol's Recording Water Gauge.

particular ranges and are the result of many years' experience. For ranges of less than 12 ft. of water, the diaphragm form of pressure

FIG. 346.

FIG. 347.

tube (Fig. 346) is usually employed, while the helical form (Fig. 347) is used for higher ranges.

The bulb casting, which is of strong construction, may be placed, as an extra protection, inside of a perforated barrel or keg. If desired, the connecting tube may be put, for additional safety, in an iron pipe conduit.

When the recorder is placed outdoors, it is protected by a moisture-

FIG. 348. — Moisture-proof Cast Iron Case.

proof cast iron case (Fig. 348) in which the interior parts of the recording instrument are mounted on a skeleton back. For weekly

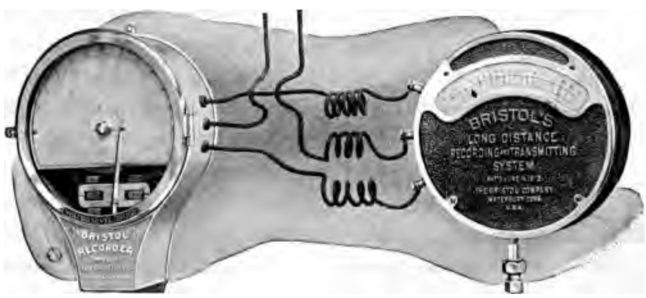


FIG. 349. — Bristol's Long-distance Recorder.

records, the chart is marked as shown in Fig. 345, the dark portions being used for the records made at night.

566. *Bristol's Long-distance Electric Transmitting and Recording System.* — When equipped with an electric transmitting system (Fig. 349), the Bristol gauges make continuous automatic records of variations in water level, etc., at long distances. The transmitting and recording instruments are connected by three wires. With a 110-volt 60-cycle alternating current circuit, and No. 14 copper wires, the recorder may be installed 3 miles from the transmitter, and with No. 12 wire, this distance can be increased to 5 miles. When direct current circuits are used, a small motor generator set is required, in connection with the transmitting.

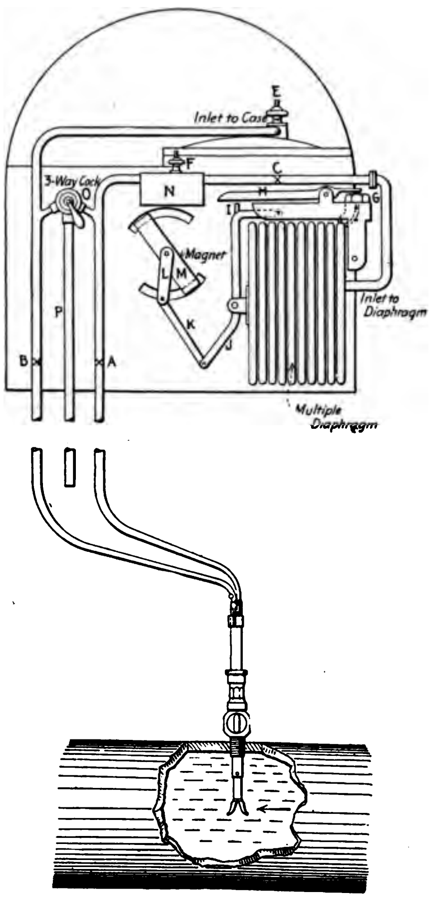


FIG. 350. — The Cole Recorder.

567. The Cole Recorder (Fig. 350) is a sensitive station meter which records on a chart, by means of a pen, the fluctuations in the delivery of a water pipe. In installing this meter, a careful traverse of the pipe is made with a rod-meter (p. 489) to ascertain the ratio of the average velocity of flow in the pipe to the center velocity. The rod-meter is then set permanently with its orifices at the center of the pipe, and its Pitot tubes are connected with the recorder by small pressure pipes, provided with stop-cocks. These pipes and a waste-pipe for expelling air are attached to a 3-way cock which controls the admission of water to the recorder. A multiple diaphragm of ten sections, placed in the recorder, is rigidly attached to the inlet-head of the case. When water from the pressure pipe of the up stream Pitot tube is admitted, it expands the diaphragm, the amount of expansion depending upon the velocity of the water in the pipe. By a suitable arrangement of magnets and links, the diaphragm rotates a spiral or cam which revolves freely on pivots, without doing any work. The spiral simply limits the movement of the pen-arm which is lifted and lowered at one-minute intervals, and thus made to mark on a chart revolved by clockwork the fluctuation in the rate of flow. The charts are usually laid out for gallons per 24 hours, but any other unit may be adopted. The spiral is determined by actual calibration to give pen ordinates directly proportional to the flow.

568. The Hydro-chronograph* (Fig. 351) gives a complete graphic record of the varying head of streams, reservoirs, or other bodies of water. It is operated by a glass float, attached to a brass link-chain, which passes over a sprocket-wheel on the left-hand side of the upper frame of the instrument, the other end of the chain being connected to an iron counterweight, balancing the dead weight of the float, when it is partially immersed in water. As the float rises or falls with the water, the sprocket-wheel revolves, raising or lowering the recording pencils by means of small bevel-gears, acting on a vertical worm and nut. As the drum is revolved by clockwork, the pencil traces a curve on the chart, each point of which corresponds to the time as laid off on the chart.

FIG. 351. — Hydro-chronograph.

The instrument is made entirely of brass, all finished surfaces being burnished and the remainder black-lacquered. Each instrument is provided with a weather-proof teakwood case.

In installing the apparatus, the instrument is set on a shelf or platform at a point above the surface of the water. A 3-in. pipe, in which the glass float travels, is set vertically beneath the instrument. This pipe must extend from a point below the lowest water line to a point above the highest. Holes are drilled in it below the water lines, for free admission of the water.

The hydro-chronograph has no springs, diaphragms, or other delicate parts, to get out of order. The glass float cannot become corroded or affected by temperature or chemical action. The record chart, which is larger than that of other similar instruments, is rectangular, making the divisions regular for all heights of water and for all periods of time.

* Made by Hydro Mfg. Co., Philadelphia, Pa.

569. Friez's automatic water stage register * (Fig. 352) connected with a float makes automatically an accurate graphic record of changes in the elevation of the water in a river, reservoir, tank, etc., on a suitably ruled record sheet, which can be removed for filing.

The register consists of a metal base, on which is mounted a power clock for propelling, by means of a feed-screw, a recording pen-carriage, a revolvable record drum around which the record sheet is placed, and one or more sprocket-wheels to rotate the drum. A

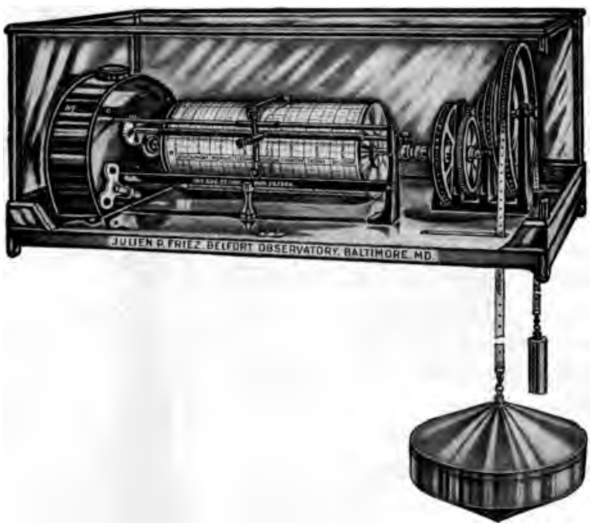


Fig. 352. — Friez's Automatic Water Stage Register.

non-corrosive, perforated, flexible metal band connects a float resting on the water surface with a suitable counterweight. This band passes over a sprocket-wheel, and its perforations engage accurately spaced pins thereon, any motion of the float up or down being thereby positively transmitted to the recording mechanism.

The record drum extends lengthwise of the register, and can be removed at will, for special examination of the record, or for putting on new sheets. The clock is placed at the left-hand end of the register, and, by means of the feed-screw, propels the pen-carriage and its recording pen lengthwise of the drum, at a fixed and uniform rate of speed, and in a perfectly straight line. The time rate provided is for daily and weekly records, or either alone. The pen-carriage can be started or stopped at will, or shifted from one position to another without stopping or disturbing the clock. The clock is,

* Manufactured by Julien P. Friez and Sons, Baltimore, Md.

also, provided with a dial having hour, minute, and second hands, plainly visible, and its cover can be removed and the entire mechanism exposed, without disturbing any of the working parts of the register or the clock, or even the hands.

By changing the arrangement of the sprocket, the ratio of one revolution of the wheel to a foot rise or fall of the water can be varied from 1 : 1 to 1 : 15 or 1 : 20.

The register is fitted with a glass cover, provided with lock and key, thus protecting the parts from dust and dirt, without interfering with the visibility of the records or operating mechanisms. Friction is reduced to a minimum, and the register is sensitive to and records the slightest changes or fluctuations of water level.

570. Stevens continuous water stage recorder* (Fig. 353) is an instrument designed to record accurately fluctuations of water level in reservoirs, tanks, sewers, etc. It acts as follows:

A roll of suitably ruled record paper is moved forward, at a uniform rate, by a power weight and a clock, while a pencil point or ink pen

FIG. 353. — Stevens Continuous Water Stage Recorder.

is moved laterally by means of a float, according to the fluctuations of the water level. The combined effect of these two motions produces the record.

The paper on which the record is made is wound accurately on a brass cylinder, called the *supply-drum*, from which it passes over the *recording drum*, and then into a suitable receptacle. The recording drum is driven at a uniform rate by the power weight through the chain over the sprocket-wheel and is released by the clock.

The pencil or pen is fixed to the carriage, which moves laterally on the track. It is driven by the tape-pin fixed to the endless per-

* Manufactured by Leupold, Voelpel & Co., Portland, Ore.

forated ribbon which runs over the spine-wheels. The left spine-wheel is fixed to the same shaft as the float pulley, which is driven by the float through a cable and counterpoise. The instrument is made with such accuracy that no *lost motion* occurs between the float and the pencil.

The recorder is provided with a special device which causes all fluctuations to be automatically recorded, even if they should exceed the limit for which the instrument is installed. This is effected by causing the pencil to reverse its direction suddenly, whenever it reaches the margin of the record paper. After the reversal, the record continues as before.

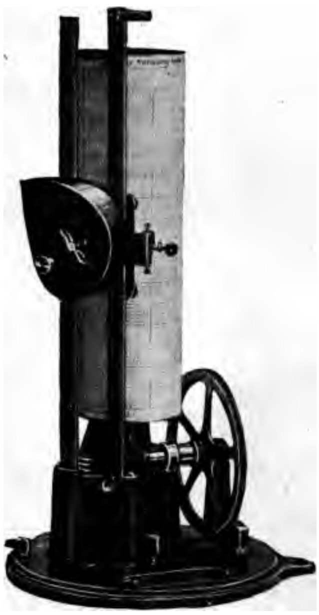


FIG. 354. — Stevens Eight-day Recording Water Gauge.

The reversal of the motion of the pencil is accomplished in a simple mechanical manner, and takes place at both margins of the paper without *lost motion*, so that a complete record of all fluctuations to the same scale is insured.

The recorder should be placed on top of a *stilling-box*, which stops all wave action. The clock will run as long as there is space to permit the power weight to move. This weight falls at a rate of 7 ft. per month.

571. *Stevens eight-day recording water gauge* (Fig. 354) has a vertical drum for carrying the record paper, which is revolved by means of a float that drives through a cable and a counterpoise, a pulley connected to the recorder, to correspond to the changes in water level. A clock is mounted to run downward on a track parallel with the drum. It carries with it a pencil or ink pen that traverses the length of the drum in 8 days. The combined motion of the pencil and the turning of the drum produces a graphical record of the changes in water level.

The clock simply serves to regulate the rate of travel of the pencil or pen along the drum. Attached to the shaft of the clock that carries the main spring is a small gear-wheel that meshes with a vertical rack. As the spring unwinds, the gear turns, allowing the clock to travel downward at a speed of $1\frac{1}{4}$ ins. per day. The weight of the clock and the pencil carriage assist the spring in driving the clock.

572. *Stevens long-distance water stage recorder* (Fig. 355) makes a record of the fluctuations in water level at any distance to which wires can be strung. The changes of water level are communicated electrically to a *Stevens continuous water stage recorder* by means of a *sender* and a *receiver*, which are suitably connected by a wire, which receives electric current from batteries.

The *sender* is mounted, approximately level, on top of a stilling-well in which a float moves up and down with the rise and fall of the

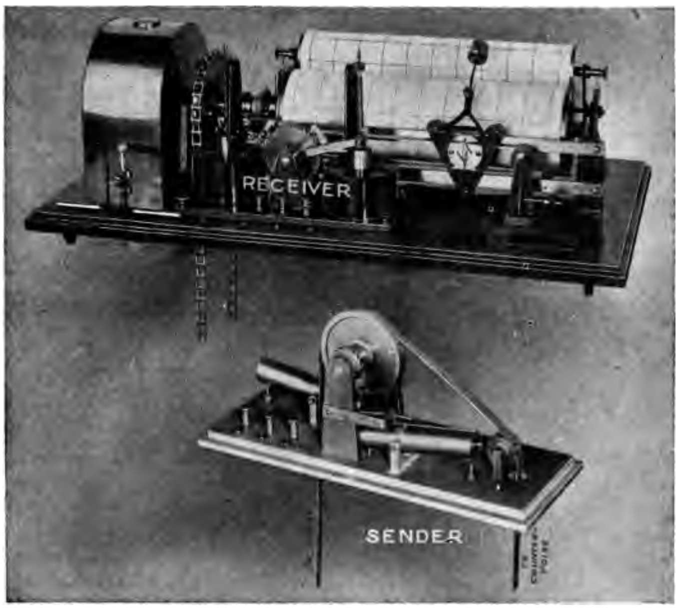


FIG. 355. — Stevens Long-distance Water Stage Recorder.

water. The *receiver* and the *recorder* are placed in any convenient place in an office, power-house, etc. The *sender* and *receiver* are connected by a single wire, the ground being used for the return current. Whenever the float in the stilling-well moves up or down, the electric circuit is closed by an ingenious device whereby a contact of nearly a second's duration is secured, such a contact being essential for the operation of the *receiver*. Except during the one second in which the water stage completes its change interval, the electric circuit is open. Hence the required electric current can be obtained from ordinary dry cells, such as are used for door bells and other kinds of intermittent electrical service.

The motion which the *sender* receives from the float is communicated by electricity to the *receiver* and actuates the *recorder*.

573. The Sanborn Recorder* (Fig. 356) consists of a recording device, the *compensator* or inlet tubing, and the connecting tubing. No well, float, or rubber diaphragm is required. The instrument is operated by air confined in the tubing, which is compressed as the water rises, and exerts a pressure on a set of corrugated non-corrosive diaphragms, which have high tensile strength and elasticity. The movement of the diaphragms is transmitted to a recording pen-arm, by a simple mechanism, consisting of two levers and an axis.

The recorder is placed at any convenient distance above the water

FIG. 356. — Sanborn Recorder.

surface, whose fluctuations in level are to be recorded. After the inlet tubing has been lowered to the proper position, the water tries to enter the open end of the cylindrical tubing, which is full of air. In entering the tubing, the water produces a *plunger* effect and exerts a pressure that moves the pen-arm of the recorder, which marks on a chart, turned by clockwork, the variations in water level.

For a range of 10 feet in fluctuations of water level, the recorder reads directly to the nearest tenth of a foot, and half-tenths can be readily

* Manufactured by the Sanborn Co., Boston, Mass.

estimated. For a range of 20 feet, the instrument reads to two-tenths, etc. It can be depended on to be accurate within 2 or 3 per cent.

This recorder has been used very successfully in the sewers of Boston and of other cities, for recording variations in the level of the water. On account of its construction, it is neither obstructed nor corroded by sewage. It is also suitable for gauging silty rivers, the run-off of drainage areas, etc.

574. The Lea V-notch Recording Meter * records graphically the flow of water over a V-notch weir, and is used for measuring the quantity of feed-water delivered to boilers, the discharge of the condensers, or the delivery of a stream, canal, sewer, etc.

Fig. 357 shows the construction of this meter where the flow fluctuates considerably. If the flow remains practically constant, a simpler type

FIG. 357. — Lea V-notch Recording Meter.

of the meter is used. The V-notch is placed in a closed tank. A large still-water chamber is provided above the weir, and a still larger space for storage extends under the still-water weir-chamber. The inlet-valve is controlled from the large storage surface below the weir, and hence the rate of change of head on the weir is very much reduced, under varying conditions.

The head of water flowing over the V-notch is measured by means of a seamless float, operating in the still-water chamber, out of the path of the flow. A spindle, rigidly attached to the float, passes through the upper part of the tank and through an anti-vapor gland in the bottom

* Manufactured by the Yarnall-Waring Co., Philadelphia, Penn.

of the *Lea Recorder*. The upper end of this spindle is formed as a pointer, which indicates on a graduated scale, as the float rises and falls, the exact flow over the V-notch in inches. A rack on the spindle gears into a small pinion, located upon the axis of a drum. Upon the drum there is a screw-thread, the contour of which is the *curve of flow* for the V-notch weir. Into this thread fits one end of a slider-bar, supported on pivoted rollers. The other end is fitted with a pencil-point or pen, in contact with a paper chart, mounted on a clock-driven drum, which revolves once in 24 hours.

By the arrangement described above, the actual depth of water in the notch can be observed at any time on a vertical scale; the rate of flow at any moment can be seen with a high degree of accuracy by glancing at the graduations on the drum; and a diagram, the area of which gives the total discharge for any period, is drawn on the chart attached to the drum. This area can be determined by means of a planimeter, but this work may be omitted, if a special integrating attachment is used, by means of which the total quantity of water delivered can be read on counter-dials at a glance.

Note. — The discharge of a V-notch meter was first investigated by Prof. James Thompson* in 1858 to 1863. Compared with a rectangular weir, the V-notch has the advantage that even with small flows the depth of the water is sufficiently large to be accurately measured. Moreover, as the ratio of depth to width is always constant in a V-notch weir, the coefficient of discharge remains practically constant.

Dr. James Barr made a very extended series of experiments upon the flow of V-notch weirs.† According to these experiments, the discharge of a V-notch weir is given by the formula:

$$Q = cH^{\frac{5}{2}},$$

where Q = discharge in cubic feet,
H = head in inches,
c = coefficient.

For a 90-degree V-notch, the heads varying from 2 to 10 inches, Dr. Barr found c to vary from 0.3104 to 0.2995.

In 1905 James E. Lea utilized the advantages of the V-notch weir by making it part of a recording instrument for boiler-feed and condensate measurements.

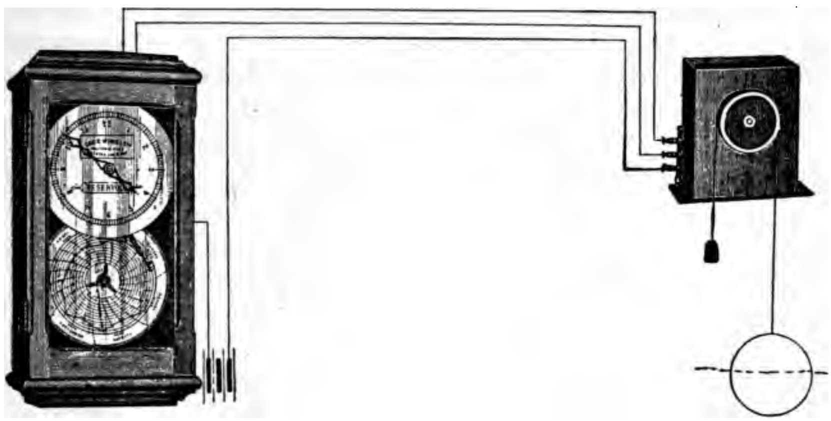
575. The Winslow apparatus ‡ for indicating and recording the depth of water in reservoirs, stand-pipes, pump-wells, etc., consists of a transmitting and an indicating device (Fig. 573*a*). The *transmitter* is inclosed in an iron case, to protect it from dust and injury. A grooved

* Professor of Civil Engineering at Queen's College, Belfast, and at Glasgow University.

† Engineering, London, April 8 and 15, 1910.

‡ Patented and manufactured by George E. Winslow, Waltham, Mass.

pulley, on which the float-line is placed, is attached to the front of the case. A float, which rises and falls with the variation of the water elevation, is attached to one end of the float-line, and a balancing weight to the other. A pipe, at least 2 inches larger in diameter than the float, should be placed to protect the float, to prevent it from being influenced by surges, currents, etc. The bottom of the pipe should be



INDICATOR
Dial, 10 Inches Diameter

TRANSMITTER

FIG. 357*a*.—Winslow Transmitting and Indicating Apparatus.

sealed, and a non-corrosive inlet, about $\frac{1}{2}$ in. in diameter, should be placed about 6 ins. above the bottom of the pipe. Oil may be put in this pipe, to keep the water below the depth at which ice might form, thus insuring the movement of the float. The motion of the float revolves the grooved pulley, which is in communication with the *indicator* through electric wires.

The *indicator*, which can be placed in any convenient office, shows the height of the water surface. It is provided with a dial, divided in feet, each of which is subdivided, the smallest division representing usually a tenth of a foot or larger. The hand upon the dial moves forward or backward, as the float rises or falls.

Below the indicator dial, another dial is placed, upon which a record of the indicating dial is traced in a diagram. The hand of the record dial makes one revolution in one or seven days, as may be desired, and has divisions indicating hours and days of the week, and also concentric circles representing feet.

An electric alarm-bell may be located at any desired point, preferably in proximity to the indicator. It will give an alarm at intervals, on movable contact on the indicating dial, thus drawing attention to the height of water in the reservoir, etc.

The connection between the *transmitter* and *indicator* is made by two, three or four wires. When two wires are used, a battery is placed at the transmitter, to control the lines, and another battery is put at the *indicatar*, to operate it through a relay. About 100 to 140 milli-amperes of current are required to operate the *indicator*. Two or three cells are needed for this purpose, in addition to the battery, to run the lines and the indicator. With the *improved transmitter*, three cells must be used to operate the *transmitter*. The *improved transmitter* is for use where a float cannot be used.

In the water works of Boston, Mass., this apparatus with two wires is in use where the distance between *transmitter* and *indicator* is 17 miles.

APPENDIX I

STANDARD SPECIFICATIONS FOR CAST IRON PIPE AND SPECIAL CASTINGS

ADOPTED BY AMERICAN WATER WORKS ASSOCIATION, *May* 12, 1908

Description of Pipes. — Section 1. The pipes shall be made with hub-and-spigot joints, and shall accurately conform to the dimensions given in Tables Nos. 1 and 2. They shall be straight and shall be true circles in section, with their inner and outer surfaces concentric, and shall be of the specified dimensions in outside diameter. They shall be at least 12 ft. in length, exclusive of socket.

Pipes with thickness and weight intermediate between the classes in Table No. 2 shall be made of the same outside diameter as the next heavier class. Pipes with thickness and weight less than shown by Table No. 2 shall be made of the same outside diameter as the Class A pipe; and pipes with thickness and weight more than shown by Table No. 2 shall be made of the same outside diameter as the Class D pipe.

All pipes having the same outside diameter shall have the same inside diameter at both ends. The inside diameter of the lighter pipes of each standard outside diameter shall be gradually increased for a distance of about 6 ins. from each end of the pipe so as to obtain the required standard thickness and weight for each size and class of pipe.

For pipes of each size from 4 ins. to 24 ins. inclusive, there shall be two standards of outside diameter, and for pipes from 30 ins. to 60 ins. inclusive, there shall be four standards of outside diameter, as shown by Table No. 1. The nominal diameters to be cast on pipes above 4 ins.

For pipes 4 ins. to 12 ins. inclusive, one class of special castings shall be furnished, made from Class D pattern. Those having spigot ends shall have outside diameters of spigot ends midway between the two standards of outside diameter as shown by Table No. 1, and shall be tapered back for a distance of 6 ins.

For pipes from 14 ins. to 24 ins. inclusive, two classes of special castings shall be furnished; Class B special castings with Classes A and B pipes, and Class D special castings with Classes C and D pipes; the former shall have cast on them the letters " AB " and the latter "CD." For pipes 30 ins. to 60 ins. inclusive, four classes of special castings shall be furnished, one for each class of pipe, and shall have cast on them the letter of the class to which they belong.

Allowable Variation in Diameter of Pipes and Sockets. — Section 2. Especial care shall be taken to have the sockets of the required size. The sockets and spigots will be tested by circular gauges, and no pipe will be received which is defective in joint room from any cause. The diameters of the sockets and the outside diameters of the spigot ends of the pipes shall not vary from the standard dimensions by more than 0.06 of an inch for pipes 16 ins. or less in diameter; 0.08 of an inch for 18-in., 20-in. and 24-in. pipes; 0.10 of an inch for 30-in., 36-in. and 42-in. pipes; 0.12 of an inch for 48-in., and 0.15 of an inch for 54-in. and 60-in. pipes.

Allowable Variation for Thickness. — Section 3. For pipes whose standard thickness is less than 1 in., the thickness of metal in the body of the pipe shall not be more than 0.08 of an inch less than the standard thickness, and for pipes whose standard thickness is 1 in. or more, the variation shall not exceed 0.10 of an inch, except that for spaces not exceeding 8 ins. in length in any direction, variations from the standard thickness of 0.02 of an inch in excess of the allowance above given shall be permitted.

For special castings of standard patterns a variation of 50 per cent greater than allowed for straight pipes shall be permitted.

Defective Spigots May be Cut. — Section 4. Defective spigot ends on pipes 12 ins. or more in diameter may be cut off in a lathe and a half-round wrought-iron band shrunk into a groove cut in the end of the pipe. Not more than 12 per cent of the total number of accepted pipes of each size shall be cut and banded, and no pipe shall be banded which is less than 11 ft. in length, exclusive of the socket.

In case the length of a pipe differs from 12 ft., the standard weight of the pipe given in Table No. 2 shall be modified in accordance therewith.

Special Castings. — Section 5. All special castings shall be made in accordance with the cuts and the dimensions given in the tables forming a part of these specifications.

The diameters of the sockets and the external diameters of the spigot ends of the special castings shall not vary from the standard dimensions by more than 0.12 of an inch for castings 16 ins. or less in diameter; 0.15 of an inch for 18 ins., 20 ins. and 24 ins.; 0.20 of an inch for 30 ins., 36 ins. and 42 ins., and 0.24 of an inch for 48 ins., 54 ins. and 60 ins. These variations apply only to special castings made from standard patterns.

The flanges on all manhole castings and manhole covers shall be faced true and smooth, and drilled to receive bolts of the sizes given in the tables. The manufacturer shall furnish and deliver all bolts for bolting on the manhole covers, the bolts to be of the sizes shown on plans and made of the best quality of mild steel, with hexagonal heads and nuts and sound, well-fitting threads.

Marking. — Section 6. Every pipe and special casting shall have distinctly cast upon it the initials of the maker's name. When cast especially to order, each pipe larger than 4 ins. may also have cast upon it figures showing the year in which it was cast and a number signifying the order in point of time in which it was cast, the figures denoting the year being above and the number below, thus:

1908	1908	1908
1	2	3

etc., also any initials, not exceeding four, which may be required by the purchaser. The letters and figures shall be cast on the outside and shall not be less than 2 ins. in length and $\frac{1}{8}$ of an inch in relief for pipes 8 ins. in diameter and larger. For smaller sizes of pipes the letters may be 1 in. in length. The weight and the class letter shall be conspicuously painted in white on the inside of each pipe and special casting after the coating has become hard.

Allowable Percentage of Variation in Weight. — Section 7. No pipe shall be accepted the weight of which shall be less than the standard weight by more than 5 per cent, for pipes 16 ins. or less in diameter, and 4 per cent for pipes more than 16 ins. in diameter, and no excess above the standard weight of more than the given percentage for the several sizes shall be paid for. The total weight to be paid for shall not exceed for each size and class of pipe received the sum of the standard weights of the same number of pieces of the given size and class by more than 2 per cent.

No special casting shall be accepted the weight of which shall be less than the standard weight by more than 10 per cent for pipes 12 ins. or less in diameter, and 8 per cent for larger sizes, except that curves, Y pieces and breeches pipe may be 12 per cent below the standard weight, and no excess above the standard weight of more than the above percentages for the several sizes will be paid for. These variations apply only to castings made from the standard patterns.

Quality of Iron. — Section 8. All pipes and special castings shall be made of cast iron of good quality, and of such character as shall make the metal of the castings strong, tough and of even grain, and soft enough to satisfactorily admit of drilling and cutting. The metal shall be made without any admixture of cinder iron or other inferior metal, and shall be remelted in a cupola or air furnace.

The contractor shall have the right to make and break three bars from each heat or run of metal, and the test shall be based upon the average results of the three bars. Should the dimensions of the three bars differ from those given below, a proper allowance therefor shall be made in the results of the tests.

Tests of Material. — * Section 9. Specimen bars of the metal used, each being twenty-six inches long by two inches wide and one inch thick, shall be made without charge as often as the engineer may direct, and in default of definite instructions, the contractor shall make and test at least one bar from each heat or run of metal. The bars when placed flatwise upon supports twenty-four inches apart, and loaded in the center, shall support a load of 1900 lbs. and show a deflection of not less than 0.30 of an inch before breaking; or if preferred, tensile bars shall be made which will show a breaking point of not less than 19,000 lbs. per square inch.

Casting of Pipe. — Section 10. The straight pipes shall be cast in dry sand molds in a vertical position. Pipes 16 ins. or less in diameter shall be cast with the hub end up or down, as specified in the proposals. Pipes 18 ins. or more in diameter shall be cast with the hub end down.

The pipes shall not be stripped or taken from the pit while showing color of heat, but shall be left in the flasks for a sufficient length of time to prevent unequal contraction by subsequent exposure.

Quality of Castings. — Section 11. The pipes and special castings shall be smooth, free from scales, lumps, blisters, sand holes and defects of every nature which make them unfit for the use for which they are intended. No plugging or filling will be allowed.

Cleaning and Inspection. — Section 12. All pipes and special castings shall be thoroughly cleaned and subjected to a careful hammer inspection. No casting shall be coated unless entirely clean and free from rust, and approved in these respects by the engineer immediately before being dipped.

Coating. — Section 13. Every pipe and special casting shall be coated inside and out with coal-tar pitch varnish. The varnish shall be made from coal tar. To this material sufficient oil shall be added to make a smooth coating, tough and tenacious when cold, and not brittle nor with any tendency to scale off.

* Pipe may be made under higher metal tests when desired. Stock pipe may be made under metal tests as low as 1800 lbs.

Each casting shall be heated to a temperature of 300 degrees Fahrenheit immediately before it is dipped, and shall possess not less than this temperature at the time it is put in the vat. The ovens in which the pipes are heated shall be so arranged that all portions of the pipe shall be heated to an even temperature. Each casting shall remain in the bath at least five minutes.

The varnish shall be heated to a temperature of 300 degrees Fahrenheit (or less if the engineer shall so order), and shall be maintained at this temperature during the time the casting is immersed.

Fresh pitch and oil shall be added when necessary to keep the mixture at the proper consistency, and the vat shall be emptied of its contents and refilled with fresh pitch when deemed necessary by the engineer. After being coated the pipe shall be carefully drained of the surplus varnish. Any pipe or special casting that is to be recoated shall first be thoroughly scraped and cleaned.

Hydrostatic Test. — Section 14. When the coating has become hard, the straight pipes shall be subjected to a proof by hydrostatic pressure, and, if required by the engineer, they shall also be subjected to a hammer test under this pressure.

The pressure to which the different sizes and classes of pipes shall be subjected are as follows:

	20 ins. diameter and larger, pounds per square inch	Less than 20 ins. diameter, pounds per square inch
Class A Pipe	150	300
Class B Pipe	200	300
Class C Pipe	250	300
Class D Pipe	300	300

Weighing. — Section 15. The pipes and special castings shall be weighed for payment under the supervision of the engineer after the application of the coal-tar pitch varnish. If desired by the engineer, the pipes and special castings shall be weighed after their delivery, and the weights so ascertained shall be used in the final settlement, provided such weighing is done by a legalized weighmaster. Bids shall be submitted and a final settlement made upon the basis of a ton of 2000 lbs.

Contractor to Furnish Men and Materials. — Section 16. The contractor shall provide all tools, testing machines, materials and men necessary for the required testing, inspection and weighing at the foundry of the pipe and special castings; and should the purchaser have no inspector at the works, the contractor shall, if required by the engineer, furnish a sworn statement that all of the tests have been made as specified, this statement to contain the results of the tests upon the test bars.

Power of Engineer to Inspect. — Section 17. The engineer shall be at liberty at all times to inspect the material at the foundry, and the molding, casting and coating of the pipes and special castings. The forms, sizes, uniformity and condition of all pipes and other castings herein referred to shall be subject to his inspection and approval, and he may reject, without proving, any pipe or other casting which is not in conformity with the specifications or drawings.

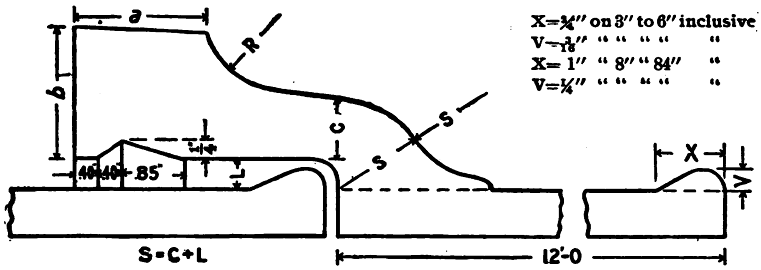
Inspector to Report. — Section 18. The inspector at the foundry shall report daily to the foundry office all pipes and special castings rejected, with the causes for rejection.

Castings to be Delivered Sound and Perfect. — Section 19. All the pipes and other castings must be delivered in all respects sound and conformable to these specifications. The inspection shall not relieve the contractor of any of his obligations in this respect, and any defective pipes or other castings which may have passed the engineer at the works or elsewhere shall be at all times liable to rejection when discovered, until the final completion and adjustment of the contract; provided, however, that the contractor shall not be held liable for pipes or special castings found to be cracked after they have been accepted at the agreed point of delivery. Care shall be taken in handling the pipes not to injure the coating, and no pipes or other material of any kind shall be placed in the pipes during transportation or at any time after they have received the coating.

Definition of the Word "Engineer." — Section 20. Wherever the word "engineer" is used herein it shall be understood to refer to the engineer or inspector acting for the purchaser and to his properly authorized agents, limited by the particular duties intrusted to them.

STANDARD DIMENSIONS OF PIPE

Table No. 1 **Classes A, B, C, D**



Nominal Diam. Inches	Classes	Actual Outside Diam. Inches	Diam. of Sockets: Pipe Inches	Diam. of Sockets: Special Castings Inches	Depth of Sockets: Pipe Inches	Depth of Sockets: Special Castings Inches	A	B	C
4	A	4.80	5.60	5.70	3.50	4.00	1.5	1.30	.65
4	B—C—D	5.00	5.80	5.70	3.50	4.00	1.5	1.30	.65
6	A	6.90	7.70	7.80	3.50	4.00	1.5	1.40	.70
6	B—C—D	7.10	7.90	7.80	3.50	4.00	1.5	1.40	.70
8	A—B	9.05	9.85	10.00	4.00	4.00	1.5	1.50	.75
8	C—D	9 30	10.10	10.00	4.00	4.00	1.5	1.50	.75
10	A—B	11.10	11.90	12.10	4.00	4 00	1.5	1.50	.75
10	C—D	11.40	12.20	12.10	4.00	4.00	1.5	1.60	.80
12	A—B	13.20	14.00	14.20	4.00	4.00	1.5	1.60	.80
12	C—D	13.50	14.30	14.20	4.00	4.00	1.5	1.70	.85
14	A—B	15.30	16.10	16.10	4.00	4.00	1.5	1.70	.85
14	C—D	15.65	16.45	16.45	4.00	4.00	1.5	1.80	.90
16	A—B	17.40	18.40	18.40	4.00	4.00	1.75	1.80	.90
16	C—D	17.80	18.80	18.80	4.00	4.00	1.75	1.90	1.00
18	A—B	19.50	20.50	20.50	4.00	4.00	1.75	1.90	.95
18	C—D	19.92	20.92	20.92	4.00	4.00	1.75	2.10	1.05
20	A—B	21.60	22.60	22.60	4.00	4.00	1.75	2.00	1.00
20	C—D	22.06	23 06	23.06	4.00	4.00	1.75	2.30	1.15
24	A—B	25.80	26.80	26.80	4.00	4.00	2 00	2.10	1.05
24	C—D	26.32	27.32	27.32	4.00	4.00	2.00	2.50	1.25
30	A	31.74	32.74	32.74	4.50	4.50	2.00	2.30	1.15
30	B	32.00	33.00	33,00	4.50	4.50	2.00	2.30	1.15
30	C	32.40	33.40	33.40	4.50	4.50	2.00	2.60	1.32
30	D	32.74	33.74	33.74	4.50	4.50	2.00	3.00	1.50
36	A	37.96	38.96	38.96	4.50	4.50	2.00	2.50	1.25
36	B	38.30	39.30	39.30	4.50	4.50	2.00	2.80	1.40
36	C	38.70	39.70	39.70	4.50	4.50	2.00	3.10	1.60
36	D	39.16	40.16	40.16	4.50	4.50	2.00	3.40	1.80
42	A	44.20	45.20	45.20	5.00	5.00	2.00	2.80	1.40
42	B	44.50	45.50	45.50	5.00	5.00	2.00	3.00	1.50
42	C	45.10	46.10	46.10	5.00	5.00	2.00	3 40	1.75
42	D	45.58	46.58	46.58	5.00	5.00	2.00	3.80	1.95
48	A	50.50	51.50	51.50	5.00	5.00	2.00	3.00	1.50
48	B	50.80	51.80	51.80	5.00	5.00	2.00	3.30	1.65
48	C	51.40	52.40	52.40	5.00	5.00	2.00	3.80	1.95
48	D	51.98	52.98	52.98	5.00	5.00	2.00	4.20	2.20
54	A	56.66	57.66	57.66	5.50	5.50	2.25	3.20	1.60
54	B	57.10	58.10	58.10	5.50	5.50	2.25	3.60	1.80
54	C	57.80	58.80	58.80	5.50	5.50	2.25	4.00	2.15
54	D	58.40	59.40	59.40	5.50	5.50	2.25	4.40	2.45
60	A	62.80	63.80	63.80	5.50	5.50	2.25	3.40	1.70
60	B	63.40	64.40	64.40	5.50	5.50	2.25	3.70	1.90
60	C	64.20	65.20	65.20	5.50	5.50	2.25	4.20	2.25
60	D	64.82	65.82	65.82	5.50	5.50	2.25	4.70	2.60
72	A	75.34	76.34	76.34	5.50	5 50	2.25	3.80	1.87
72	B	76.00	77.00	77.00	5.50	5.50	2.25	4.20	2.20
72	C	76.88	77.88	77.88	5.50	5.50	2.25	4.60	2.64
84	A	87.54	88.54	88.54	5.50	5.50	2.50	4.10	2.10
84	B	88 54	89.54	89.54	5 50	5.50	2.50	4.50	2 60

STANDARD THICKNESS AND WEIGHTS OF CAST IRON PIPE

Table No. 2 Classes A, B, C, D

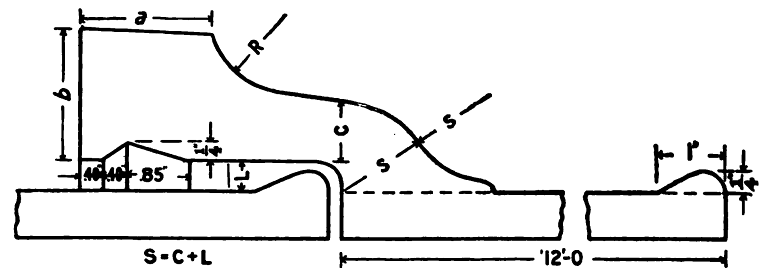
Nominal Inside Diameter Inches	CLASS A 100-Feet Head 43 Pounds Pressure			CLASS B 200-Feet Head 86 Pounds Pressure			CLASS C 300-Feet Head 130 Pounds Pressure			CLASS D 400-Feet Head 173 Pounds Pressure			Nominal Inside Diameter Inches
	Thickness Inches	Weight per Foot	Weight per Length	Thickness Inches	Weight per Foot	Weight per Length	Thickness Inches	Weight per Foot	Weight per Length	Thickness Inches	Weight per Foot	Weight per Length	
4	.42	20.0	240	.45	21.7	260	.48	23 3	280	.52	25.0	300	4
6	.44	30.8	370	.48	33.3	400	.51	35 8	430	.55	38.3	460	6
8	.46	42.9	515	.51	47.5	570	.56	52.1	625	.60	55.8	670	8
10	.50	57.1	685	.57	63.8	765	.62	70.8	850	.68	76.7	920	10
12	.54	72.5	870	.62	82.1	985	.68	91.7	1100	.75	100.0	1200	12
14	.57	89.6	1075	.66	102.5	1230	.74	116.7	1400	.82	129 2	1550	14
16	.60	108.3	1300	.70	125.0	1500	.80	143.8	1725	.89	158.3	1900	16
18	.64	129.2	1550	.75	150 0	1800	.87	175.0	2100	.96	191.7	2300	18
20	.67	150 0	1800	.80	175.0	2100	.92	208.3	2500	1.03	229.2	2750	20
24	.76	204.2	2450	.89	233.3	2800	1.04	279.2	3350	1.16	306.7	3680	24
30	.88	291.7	3500	1.03	333.3	4000	1.20	400.0	4800	1.37	450.0	5400	30
36	.99	391.7	4700	1.15	454.2	5450	1.36	545.8	6550	1.58	625.0	7500	36
42	1.10	512.5	6150	1.28	591.7	7100	1.54	716.7	8600	1.78	825.0	9900	42
48	1.26	666.7	8000	1.42	750.0	9000	1.71	908.3	10900	1.96	1050.0	12600	48
54	1.35	800.0	9600	1.55	933.3	11200	1.90	1141.7	13700	2.23	1341.7	16100	54
60	1.39	916.7	11000	1.67	1104.2	13250	2.00	1341.7	16100	2.38	1583.3	19000	60
72	1.62	1283.4	15400	1.95	1545.8	18550	2.39	1904.2	22860	. .	. .	. .	72
84	1.72	1633.4	19600	2.22	2104.2	25250			. .	. .	. .	. .	84

The above weights are per length to lay 12 feet, including standard sockets; proportionate allowance to be made for any variation.

STANDARD DIMENSIONS OF PIPE

High Pressure Service

Table No. 3 **Classes E, F, G, H**



Nominal Diam. Inches	Classes	Actual Outside Diam. Inches	Diam. of Sockets — Pipe and Specials	Depth of Sockets — Pipe and Specials	A	B	C	R	Nominal Diam. Inches
6	E—F	7.22	8.02	4.00	1.50	1.75	.75	1.10	6
6	G—H	7.38	8.18	4.00	1.50	1.85	.85	1.10	6
8	E—F	9.42	10.22	4.00	1.50	1.85	.85	1.10	8
8	G—H	9.60	10.40	4.00	1.50	1.95	.95	1.10	8
10	E—F	11.60	12.40	4.50	1.75	1.95	.95	1.10	10
10	G—H	11.84	12.64	4.50	1.75	2.05	1.05	1.10	10
12	E—F	13.78	14.58	4.50	1.75	2.05	1.05	1.10	12
12	G—H	14.08	14.88	4.50	1.75	2.20	1.20	1.10	12
14	E—F	15.98	16.78	4.50	2.00	2.15	1.15	1.10	14
14	G—H	16.32	17.12	4.50	2.00	2.35	1.35	1.10	14
16	E—F	18.16	18.96	4.50	2.00	2.30	1.25	1.15	16
16	G—H	18.54	19.34	4.50	2.00	2.55	1.45	1.15	16
18	E—F	20.34	21.14	4.50	2.25	2.45	1.40	1.15	18
18	G—H	20.78	21.58	4.50	2.25	2.75	1.65	1.15	18
20	E—F	22.54	23.34	4.50	2.25	2.55	1.50	1.15	20
20	G—H	23.02	23.82	4.50	2.25	2.85	1.75	1.20	20
24	E—F	26.90	27.90	5.00	2.25	2.85	1.70	1.20	24
30	E	33.10	34.10	5.00	2.25	3.25	1.80	1.50	30
30	F	33.46	34.46	5.00	2.25	3.50	2.00	1.55	30
36	E	39.60	40.60	5.00	2.25	3.70	2.05	1.70	36
36	F	40.04	41.04	5.00	2.25	4.00	2.30	1.80	36

STANDARD THICKNESS AND WEIGHTS OF CAST IRON PIPE

For Fire Lines and Other High Pressure Service

Table No. 4 | **Classes E, F, G, H**

Nominal Inside Diameter Inches	CLASS E 500-Feet Head 217 Pounds Pressure			CLASS F 600-Feet Head 260 Pounds Pressure			CLASS G 700-Feet Head 340 Pounds Pressure			CLASS H 800-Feet Head 347 Pounds Pressure			Nominal Inside Diameter Inches
	Thickness Inches	Weight per Foot	Weight per Length	Thickness Inches	Weight per Foot	Weight per Length	Thickness Inches	Weight per Foot	Weight per Length	Thickness Inches	Weight per Foot	Weight per Length	
6	.58	41.7	500	.61	43.3	520	.65	47.1	565	.69	49.6	595	6
8	.66	61.7	740	.71	65.7	790	.75	70.8	850	.80	75.0	900	8
10	.74	86.3	1035	.80	92.1	1105	.86	100.9	1210	.92	106.7	1280	10
12	.82	113.8	1365	.89	122.1	1465	.97	135.4	1625	1.04	143.8	1725	12
14	.90	145.0	1740	.99	157.5	1890	1.07	174.2	2090	1.16	186.7	2240	14
16	.98	179.6	2155	1.08	195.4	2345	1.18	219.2	2620	1.27	232.5	2790	16
18	1.07	220.4	2645	1.17	238.4	2860	1.28	267.1	3205	1.39	286.7	3440	18
20	1.15	263.0	3155	1.27	286.3	3435	1.39	320.8	3850	1.51	344.6	4135	20
24	1.31	359.6	4315	1.45	392.9	4715	..	..	..	..	..	..	24
30	1.55	521.7	6260	1.73	585.4	7025	..	..	..	..	..	..	30
36	1.80	725.0	8700	2.02	820.0	9840	..	..	..	..	..	..	36

The above weights are per length to lay 12 feet, including standard sockets; proportionate allowance to be made for any variation.

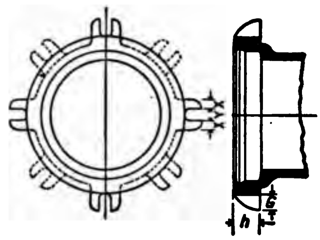
For HIGH PRESSURE PIPE from 6 inches to 24 inches inclusive, one class of special castings shall be furnished for Classes E and F pipe, and one class of special castings for Classes G and H pipe. For 30-inch and 36-inch pipe, one class of special castings shall be furnished for each class of pipe.

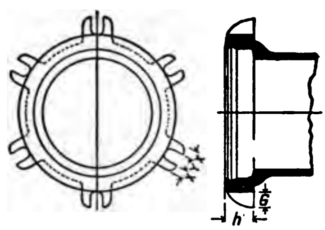
STANDARD LUGS

Number and Weights of Lugs on Outlets of Different Sizes

Table No. 5

4 Lugs, 12-14 inches
8 Lugs, 42-60 inches





6 Lugs, 16-36 inches

Nominal Diameter Outlet Inches	Number of Pairs of Lugs	Approximate Weight Lugs on One Bell Pounds	Nominal Diameter Outlet Inches	Number of Pairs of Lugs	Approximate Weight Lugs on One Bell Pounds
12	4	32	30	6	80
14	4	32	36	6	80
16	6	56	42	8	111
18	6	56	48	8	114
20	6	56	54	8	134
24	6	56	60	8	137

Two pairs of lugs are placed on the vertical axis of each bell, the others at equal distances around circumference. H is equal depth of bell on all sizes.

G equals 2.50 inches, X equals 1.25 inches, Y equals 1.63 inches for 12 to 24 inches inclusive.

G equals 3.00 inches, X equals 1.50 inches, Y equals 2.00 inches for 30 to 60 inches inclusive.

STANDARD SPECIAL CASTINGS FOR WATER

Standard Curves, Bell and Spigot, ¼, ⅛, 1/16

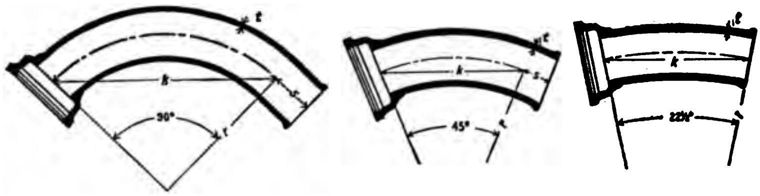


TABLE No. 6

¼ Curves					
Nominal Diam., In.	Class	Dimensions, Inches t	r	k	Approx. Weight Pounds
4	D	.52	16	22.60	82
6	D	.55	16	22.60	130
8	D	.60	16	22.60	200
10	D	.68	16	22.60	278
12	D	.75	16	22.60	366
14	B	.66	18	25.50	406
14	D	.82	18	25.50	504
16	B	.70	24	34.00	594
16	D	.89	24	34.00	750
18	B	.75	24	34.00	710
18	D	.96	24	34.00	888
20	B	.80	24	34.00	840
20	D	1.03	24	34.00	1070
24	B	.89	30	42.40	1290
24	D	1.16	30	42.40	1656
30	A	.88	36	50.90	1814
30	B	1.03	36	50.90	2082
30	C	1.20	36	50.90	2454
30	D	1.37	36	50.90	2836
36	A	.99	48	67.90	2964
36	B	1.15	48	67.90	3500
36	C	1.36	48	67.90	4120
36	D	1.58	48	67.90	4820

TABLE No. 7

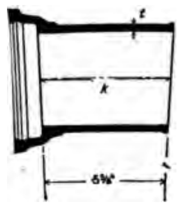
⅛ Curves						1/16 Curves		
Nominal Diam., In.	Class	t	r	k	Approx. Weight Pounds	r	k	Approx. Weight Pounds
4	D	.52	24	18.40	66	48	18.70	66
6	D	.55	24	18.40	105	48	18.70	105
8	D	.60	24	18.40	150	48	18.70	150
10	D	.68	24	18.40	202	48	18.70	202
12	D	.75	24	18.40	265	48	18.70	265
14	B	.66	36	27.60	359	72	28.10	312
14	D	.82	36	27.60	442	72	28.10	382
16	B	.70	36	27.60	445	72	28.10	388
16	D	.89	36	27.60	558	72	28.10	484
18	B	.75	36	27.60	533	72	28.10	464
18	D	.96	36	27.60	663	72	28.10	574
20	B	.80	48	36.70	758	96	37.50	676
20	D	1.03	48	36.70	964	96	37.50	858
24	B	.89	60	45.90	1181	120	46.80	1072
24	D	1.16	60	45.90	1515	120	46.80	1372
30	A	.88	60	45.90	1475	120	46.80	1342
30	B	1.03	60	45.90	1684	120	46.80	1528
30	C	1.20	60	45.90	1983	120	46.80	1800
30	D	1.37	60	45.90	2291	120	46.80	2080
36	A	.99	90	68.90	2472	180	70.20	2472
36	B	1.15	90	68.90	2916	180	70.20	2916
36	C	1.36	90	68.90	3430	180	70.20	3430
36	D	1.58	90	68.90	4012	180	70.20	4012
42	A	1.10	90	68.90	3286	180	70.20	3286
42	B	1.28	90	68.90	3778	180	70.20	3778
42	C	1.54	90	68.90	4600	180	70.20	4600
42	D	1.78	90	68.90	5360	180	70.20	5360
48	A	1.26	90	68.90	4230	180	70.20	4230
48	B	1.42	90	68.90	4820	180	70.20	4820
48	C	1.71	90	68.90	5796	180	70.20	5796
48	D	1.96	90	68.90	6750	180	70.20	6750
54	A	1.35	90	68.90	5180	180	70.20	5180
54	B	1.55	90	68.90	5990	180	70.20	5990
54	C	1.90	90	68.90	7330	180	70.20	7330
54	D	2.23	90	68.90	8620	180	70.20	8620
60	A	1.39	90	68.90	5990	180	70.20	5990
60	B	1.67	90	68.90	7130	180	70.20	7130
60	C	2.00	90	68.90	8590	180	70.20	8590
60	D	2.38	90	68.90	10240	180	70.20	10240

S = 8 inches on sizes 4 and 6 inches.
S = 10 inches on sizes 8 inches.
S = 12 inches on sizes 10 to 36 inches.

S = 6 inches on ⅛ Curves on sizes 4 to 30 inches inclusive.
S = 6 inches on 1/16 Curves on sizes 4 to 12 inches inclusive.
All weights are approximate.

STANDARD SPECIAL CASTINGS FOR WATER
Standard Curves, Bell and Spigot—Standard Offsets

$\frac{1}{32}$



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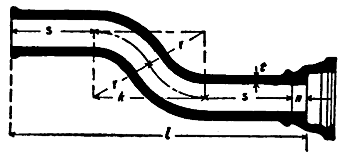


TABLE No. 8

Nominal Diam., In.	Class	t	$\frac{1}{32}$ Curves r	$\frac{1}{32}$ Curves k	$\frac{1}{32}$ Curves Approx. Weight Pounds	$\frac{1}{64}$ Curves r	$\frac{1}{64}$ Curves k	$\frac{1}{64}$ Curves Approx. Weight Pounds
4	D	.52	120	23.52	66			
6	D	.55	120	23.52	104			
8	D	.60	120	23.52	150			
10	D	.68	120	23.52	192			
12	D	.75	120	23.52	250			
14	B	.66	180	35.28	364			
14	D	.82	180	35.28	450			
16	B	.70	180	35.28	453			
16	D	.89	180	35.28	570			
18	B	.75	180	35.28	542			
18	D	.96	180	35.28	674			
20	B	.80	240	47.05	808	480	47.10	808
20	D	1.03	240	47.05	1028	480	47.10	1028
24	B	.89	240	47.05	1080	480	47.10	1080
24	D	1.16	240	47.05	1380	480	47.10	1380
30	A	.88	240	47.05	1350	480	47.10	1350
30	B	1.03	240	47.05	1540	480	47.10	1540
30	C	1.20	240	47.05	1810	480	47.10	1810
30	D	1.37	240	47.05	2090	480	47.10	2090
36	A	.99	240	47.05	1790	480	47.10	1790
36	B	1.15	240	47.05	2100	480	47.10	2100
36	C	1.36	240	47.05	2470	480	47.10	2470
36	D	1.58	240	47.05	2880	480	47.10	2880
42	A	1.10	240	47.05	2380	480	47.10	2380
42	B	1.28	240	47.05	2720	480	47.10	2720
42	C	1.54	240	47.05	3310	480	47.10	3310
42	D	1.78	240	47.05	3850	480	47.10	3850
48	A	1.26	240	47.05	3150	480	47.10	3150
48	B	1.42	240	47.05	3480	480	47.10	3480
48	C	1.71	240	47.05	4170	480	47.10	4170
48	D	1.96	240	47.05	4860	480	47.10	4860
54	A	1.35	240	47.05	3750	480	47.10	3750
54	B	1.55	240	47.05	4330	480	47.10	4330
54	C	1.90	240	47.05	5290	480	47.10	5290
54	D	2.23	240	47.05	6220	480	47.10	6220
60	A	1.39	240	47.05	4340	480	47.10	4340
60	B	1.67	240	47.05	5140	480	47.10	5140
60	C	2.00	240	47.05	6200	480	47.10	6200
60	D	2.38	240	47.05	7400	480	47.10	7400

TABLE No. 9

Nominal Diam., Inches	Class	r	l	Approx. Weight Pounds
4	D	8	35.85	91
6	D	14	46.25	183
8	D	15	48.00	280
10	D	16	49.70	390
12	D	17	51.45	530
14	B	18	53.70	555
14	D	18	53.70	695
16	B	19	55.40	708
16	D	19	55.40	900

Nominal Diam., Inches	Class	t	k	s	n
4	D	.52	13.85	10.00	2.00
6	D	.55	24.25	10.00	2.00
8	D	.60	26.00	10.00	2.00
10	D	.68	27.70	10.00	2.00
12	D	.75	29.45	10.00	2.00
14	B	.66	31.20	10.00	2.50
14	D	.82	31.20	10.00	2.50
16	B	.70	32.90	10.00	2.50
16	D	.89	32.90	10.00	2.50

STANDARD SPECIAL CASTINGS FOR WATER

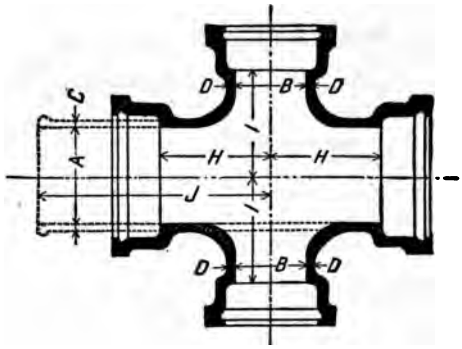


Table No. 10 Standard Branches

Nominal Diam. Inches A	Nominal Diam. Inches B	Class	Dimensions, Inches H	J	I	Approximate Weights, Pounds 3-Way Branches 2 Bells	3-Way Branches 3 Bells	4-Way Branches 3 Bells	4-Way Branches 4 Bells
4	3	D	11	23	11	121	120	153	153
4	4	D	11	23	11	125	128	164	166
6	3	D	12	24	12	173	170	207	204
6	4	D	12	24	12	185	183	223	221
6	6	D	12	24	12	203	200	259	257
8	4	D	13	25	13	262	255	301	294
8	6	D	13	25	13	278	270	333	325
8	8	D	13	25	13	301	294	378	372
10	4	D	14	26	14	356	338	395	377
10	6	D	14	26	14	371	352	424	406
10	8	D	14	26	14	389	371	461	443
10	10	D	14	26	14	414	395	511	493
12	4	D	15	27	15	473	445	514	486
12	6	D	15	27	15	486	458	540	512
12	8	D	15	27	15	502	474	573	545
12	10	D	15	27	15	519	491	605	577
12	12	D	15	27	15	540	512	651	623
14	4	B	16	28	16	485	480	535	530
14	4	D	16	28	16	614	588	666	641
14	6	B	16	28	16	500	495	560	555
14	6	D	16	28	16	634	608	730	700
14	8	B	16	28	16	515	510	600	595
14	8	D	16	28	16	662	636	787	761
14	10	B	16	28	16	535	525	635	625
14	10	D	16	28	16	679	653	822	796
14	12	B	16	28	16	560	550	680	670
14	12	D	16	28	16	698	672	860	834
14	14	B	16	28	16	575	569	723	715
14	14	D	16	28	16	750	724	938	963
16	4	B	17	29	17	615	610	675	670

STANDARD SPECIAL CASTINGS FOR WATER

Table No. 10—Continued. Standard Branches.

Nominal Diam. Inches		Class	Dimensions, Inches			Approximate Weights, Pounds			
						3-Way Branches		4-Way Branches	
A	B		H	J	I	2 Bells	3 Bells	3 Bells	4 Bells
16	4	D	17	29	17	783	760	864	841
16	6	B	17	29	17	630	625	695	690
16	6	D	17	29	17	802	779	902	879
16	8	B	17	29	17	645	640	730	725
16	8	D	17	29	17	831	808	961	938
16	10	B	17	29	17	660	655	760	755
16	10	D	17	29	17	872	849	1042	1019
16	12	B	17	29	17	685	680	805	800
16	12	D	17	29	17	884	861	1066	1043
16	14	B	17	29	17	695	690	825	820
16	14	D	17	29	17	903	880	1104	1082
16	16	B	17	29	17	729	727	904	901
16	16	D	17	29	17	991	969	1282	1259
18	4	B	18	30	18	755	750	820	815
18	4	D	18	30	18	953	927	1046	1020
18	6	B	18	30	18	765	760	840	835
18	6	D	18	30	18	968	942	1075	1049
18	8	B	18	30	18	780	775	870	865
18	8	D	18	30	18	1000	974	1140	1114
18	10	B	18	30	18	795	790	900	895
18	10	D	18	30	18	1038	1012	1216	1190
18	12	B	18	30	18	815	810	940	935
18	12	D	18	30	18	1075	1049	1290	1264
18	14	B	18	30	18	825	820	955	950
18	14	D	18	30	18	1083	1057	1306	1280
18	16	B	18	30	18	855	850	1020	1015
18	16	D	18	30	18	1108	1082	1356	1330
18	18	B	18	30	18	895	889	1101	1096
18	18	D	18	30	18	1170	1144	1480	1454
20	4	B	19	31	19	923	916	1006	999
20	4	D	19	31	19	1172	1148	1273	1248
20	6	B	19	31	19	930	920	1010	1000
20	6	D	19	31	19	1188	1164	1304	1280
20	8	B	19	31	19	945	935	1035	1025
20	8	D	19	31	19	1212	1188	1352	1328
20	10	B	19	31	19	955	945	1060	1050
20	10	D	19	31	19	1252	1227	1431	1407
20	12	B	19	31	19	975	965	1100	1090
20	12	D	19	31	19	1288	1263	1502	1479
20	14	B	19	31	19	980	970	1110	1100
20	14	D	19	31	19	1342	1318	1613	1588
20	16	B	19	31	19	1010	1000	1170	1160
20	16	D	19	31	19	1347	1323	1622	1597
20	18	B	19	31	19	1035	1025	1225	1215
20	18	D	19	31	19	1365	1341	1658	1634
20	20	B	19	31	19	1077	1070	1314	1307

STANDARD SPECIAL CASTINGS FOR WATER

Table No. 10—Continued. Standard Branches

Nominal Diam. Inches		Class	Dimensions, Inches			Approximate Weights, Pounds			
						3-Way Branches		4-Way Branches	
A	B		H	J	I	2 Bells	3 Bells	3 Bells	4 Bells
20	20	D	19	31	19	1462	1438	1852	1828
24	6	B	21	33	21	1309	1289	1425	1405
24	6	D	21	33	21	1670	1637	1809	1775
24	8	B	21	33	21	1323	1303	1453	1433
24	8	D	21	33	21	1697	1664	1863	1830
24	10	B	21	33	21	1341	1321	1489	1469
24	10	D	21	33	21	1732	1699	1933	1900
24	12	B	21	33	21	1362	1342	1532	1511
24	12	D	21	33	21	1768	1735	2005	1972
24	14	B	21	33	21	1402	1381	1609	1589
24	14	D	21	33	21	1810	1777	2088	2055
24	16	B	21	33	21	1443	1423	1694	1673
24	16	D	21	33	21	1858	1825	2185	2151
24	18	B	21	33	21	1460	1440	1727	1706
24	18	D	21	33	21	1885	1852	2238	2205
24	20	B	21	33	21	1474	1454	1756	1736
24	20	D	21	33	21	2025	1991	2518	2484
24	24	B	21	33	21	1523	1503	1854	1834
24	24	D	21	33	21	2146	2113	2727	2694
30	6	A	13	25	24	1272	1300	1407	1434
30	6	B	13	25	24	1433	1417	1580	1563
30	6	C	13	25	24	1693	1673	1870	1850
30	6	D	13	25	24	1934	1920	2113	2099
30	8	A	14	26	24	1318	1346	1453	1481
30	8	B	14	26	24	1482	1466	1624	1609
30	8	C	14	26	24	1765	1745	1953	1934
30	8	D	14	26	24	2004	1990	2182	2168
30	10	A	15	27	24	1369	1396	1512	1540
30	10	B	15	27	24	1538	1521	1685	1668
30	10	C	15	27	24	1857	1837	2075	2056
30	10	D	15	27	24	2108	2094	2319	2306
30	12	A	15	27	24	1395	1420	1555	1580
30	12	B	15	27	24	1555	1540	1715	1700
30	12	C	15	27	24	1911	1891	2184	2164
30	12	D	15	27	24	2154	2140	2411	2398
30	14	A	18	30	26	1547	1575	1737	1764
30	14	B	18	30	26	1805	1789	2085	2069
30	14	C	18	30	26	2159	2140	2497	2477
30	14	D	18	30	26	2567	2553	3026	3013
30	16	A	19	31	26	1648	1675	1805	1832
30	16	B	19	31	26	1899	1883	2200	2184
30	16	C	19	31	26	2272	2253	2662	2642
30	16	D	19	31	26	2692	2678	3206	3192
30	18	A	20	34	26	1757	1741	2024	2007
30	18	B	20	34	26	2044	1976	2387	2318

STANDARD SPECIAL CASTINGS FOR WATER

Table No. 10—Continued. Standard Branches

Nominal Diam. Inches		Class	Dimensions, Inches			Approximate Weights, Pounds			
						3-Way Branches		4-Way Branches	
A	B		H	J	I	2 Bells	3 Bells	3 Bells	4 Bells
30	18	C	20	34	26	2434	2353	2862	2781
30	18	D	20	34	26	2805	2791	3361	3348
30	20	A	21	36	26	1857	1818	2157	2118
30	20	B	21	36	26	2182	2088	2584	2490
30	20	C	21	36	26	2667	2555	3237	3126
30	20	D	21	36	26	3041	2921	3657	3538
30	24	A	23	38	26	1979	1940	2312	2274
30	24	B	23	38	26	2313	2219	2742	2648
30	24	C	23	38	26	2847	2736	3474	3362
30	24	D	23	38	26	3290	3170	4014	3895
30	30	A	26	43	26	2212	2129	2602	2520
30	30	B	26	43	26	2599	2453	3106	2960
30	30	C	26	43	26	3310	3137	4110	3937
30	30	D	26	43	26	3850	3660	4799	4609
36	8	A	14	26	27	1751	1777	1938	1963
36	8	B	14	26	27	2055	2073	2268	2287
36	8	C	14	26	27	2421	2433	2679	2691
36	8	D	14	26	27	2780	2780	3038	3039
36	10	A	15	27	27	1810	1835	1996	2021
36	10	B	15	27	27	2128	2147	2345	2364
36	10	C	15	27	27	2534	2546	2822	2834
36	10	D	15	27	27	2903	2902	3188	3188
36	12	A	16	28	27	1884	1909	2084	2109
36	12	B	16	28	27	2219	2238	2458	2477
36	12	C	16	28	27	2644	2656	2962	2973
36	12	D	16	28	27	3032	3033	3349	3350
36	14	A	18	30	29	2039	2065	2279	2304
36	14	B	18	30	29	2415	2433	2709	2728
36	14	C	18	30	29	2872	2883	3251	3263
36	14	D	18	30	29	3470	3470	4033	4033
36	16	A	19	31	29	2135	2160	2410	2436
36	16	B	19	31	29	2521	2540	2853	2872
36	16	C	19	31	29	3003	3014	3431	3442
36	16	D	19	31	29	3618	3617	4231	4230
36	18	A	20	34	29	2279	2246	2581	2548
36	18	B	20	34	29	2701	2650	3073	3022
36	18	C	20	34	29	3206	3136	3673	3604
36	18	D	20	34	29	3852	3755	4506	4409
36	20	A	21	36	29	2409	2346	2752	2689
36	20	B	21	36	29	2885	2800	3336	3251
36	20	C	21	36	29	3537	3426	4212	4101
36	20	D	21	36	29	4050	3905	4757	4612
36	24	A	23	38	29	2451	2513	2844	2907
36	24	B	23	38	29	3099	3014	2624	3539
36	24	C	23	38	29	3806	3695	4585	4474

STANDARD SPECIAL CASTINGS FOR WATER

Table No. 10—Continued. Standard Branches

Nominal Diam. Inches		Class	Dimensions, Inches			Approximate Weights, Pounds			
						3-Way Branches		4-Way Branches	
A	B		H	J	I	2 Bells	3 Bells	3 Bells	4 Bells
36	24	D	23	38	29	4511	4366	5307	5161
36	30	A	26	43	29	2830	2708	3242	3120
36	30	B	26	43	29	3594	3438	4335	4179
36	30	C	26	43	29	4248	4055	5140	4947
36	80	D	26	43	29	5160	4918	6192	5950
36	36	A	29	46	29	3067	2946	3539	3418
36	36	B	29	46	29	4046	3891	4956	4800
36	36	C	29	46	29	4788	4595	5867	5673
36	36	D	29	46	29	5810	5567	7099	6857
42	12	A	16	28	30	2507	2577	3467	3537
42	12	B	16	28	30	2670	2889	3131	3170
42	12	C	16	28	30	3478	3507	3830	3860
42	12	D	16	28	30	3971	3989	4307	4325
42	14	A	18	30	32	2671	2739	2942	3010
42	14	B	18	30	32	3075	3114	3400	3440
42	14	C	18	30	32	3747	3776	4147	4177
42	14	D	18	30	32	4590	4609	5288	5306
42	16	A	19	31	32	2778	2846	3080	3148
42	16	B	19	31	32	3196	3235	3552	3592
42	16	C	19	31	32	3891	3920	4325	4354
42	16	D	19	31	32	4754	4772	5487	5506
42	18	A	20	34	32	2950	2941	3268	3258
42	18	B	20	34	32	3407	3357	3794	3744
42	18	C	20	34	32	4393	4312	5108	5028
42	18	D	20	34	32	5049	4939	5819	5709
42	20	A	21	36	32	3104	3056	3459	3411
42	20	B	21	36	32	3582	3486	4009	3913
42	20	C	21	36	32	4615	4479	5387	5251
42	20	D	21	36	32	5297	5123	6122	5948
42	24	A	23	38	32	3314	3266	3724	3676
42	24	B	23	38	32	3852	3756	4370	4274
42	24	C	23	38	32	4965	4829	5866	5730
42	24	D	23	38	32	5709	5535	6579	6405
42	30	A	26	43	32	3679	3553	4144	4018
41	30	B	26	43	32	4554	4370	5416	5230
42	30	C	26	43	32	5649	5402	6675	6428
43	30	D	26	43	32	6561	6258	7729	7426
42	36	A	29	46	32	4076	3950	4705	4579
42	36	B	29	46	32	4903	4718	5845	5659
42	36	C	29	46	32	6150	5904	7261	7015
42	36	D	29	46	32	7187	6884	8512	8209
42	42	A	32	49	32	4393	4267	5109	4983
42	42	B	32	49	32	5533	5348	6641	6455
42	42	C	32	49	32	7001	6755	8392	8146
42	42	D	32	49	32	8158	7855	9803	9500

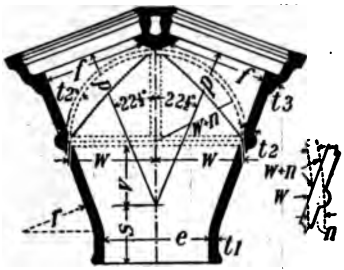
STANDARD SPECIAL CASTINGS FOR WATER

Table No. 10—Continued. Standard Branches

Nominal Diam. Inches		Class	Dimensions, Inches			Approximate Weights, Pounds			
						3-Way Branches		4-Way Branches	
A	B		H	J	I	2 Bells	3 Bells	3 Bells	4 Bells
48	12	A	17	29	33	3266	3319	3653	3707
48	12	B	17	29	33	3752	3804	4107	4160
48	12	C	17	29	33	4510	4576	4940	5007
48	12	D	17	29	33	5564	5624	6376	6436
48	14	A	18	30	35	3422	3476	3762	3815
48	14	B	18	30	35	4173	4226	4836	4889
48	14	C	18	30	35	4965	5030	5712	5778
48	14	D	18	30	35	5754	5815	6596	6656
48	16	A	19	31	35	3565	3619	3947	4001
48	16	B	19	31	35	4046	4098	4466	4519
48	16	C	19	31	35	5055	5121	5755	5821
48	16	D	19	31	35	5967	6028	6860	6921
48	18	A	20	34	35	3775	3729	4166	4120
48	18	B	20	34	35	4287	4225	4718	4655
48	18	C	20	34	35	5479	5407	6328	6256
48	18	D	20	34	35	6328	6227	7259	7158
48	20	A	21	36	35	3956	3860	4378	4282
48	20	B	21	36	35	4500	4380	4973	4853
48	20	C	21	36	35	5745	5604	6652	6511
48	20	D	21	36	35	6607	6425	7574	7392
48	24	A	23	38	35	4221	4125	4706	4609
48	24	B	23	38	35	5028	4908	5798	5678
48	24	C	23	38	35	6193	6052	7272	7131
48	24	D	23	38	35	7064	6882	7994	7812
48	30	A	26	43	35	4748	4553	5361	5166
48	30	B	26	43	35	5685	5451	6653	6418
48	30	C	26	43	35	7042	6762	8265	7985
48	30	D	26	43	35	8051	7708	9303	8960
48	36	A	29	46	35	5150	4953	5859	5662
48	36	B	29	46	35	6322	6088	7382	7148
48	36	C	29	46	35	7603	7323	8915	8635
48	36	D	29	46	35	8830	8487	10336	9993
48	42	A	32	49	35	5503	5307	6266	6069
48	42	B	32	49	35	6821	6587	7973	7739
48	42	C	32	49	35	8278	7999	9750	9470
48	42	D	32	49	35	9644	9301	11367	11024
48	48	A	35	52	35	6043	5846	7043	6846
48	48	B	35	52	35	7659	7424	9076	8841
48	48	C	35	52	35	9229	8950	11006	10726
48	48	D	35	52	35				

STANDARD SPECIAL CASTINGS FOR WATER

Standard Y Branches, Type 1



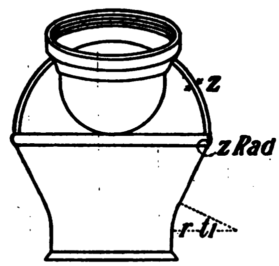
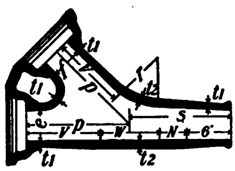


Table No. 11

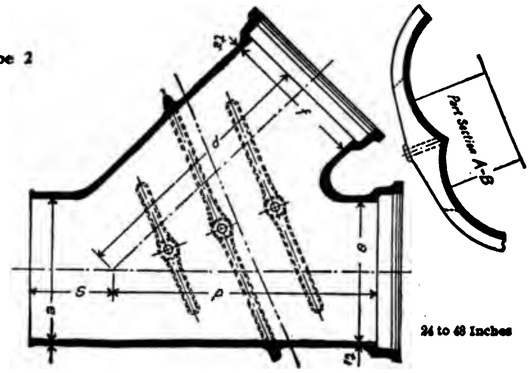
Nominal Diam., Inch. e	Nominal Diam., Inch. f	Class	s	p	v	w	n	r	Thickness, Inches t_1	Thickness, Inches t_2	Thickness, Inches t_3	Approx. Weight Pounds
12	12	D	16.0	21.50	8.00	9.79	1.17	30	75	1.08	.75	687
14	14	B	16.0	24.00	9.0	11.30	1.08	30	.66	.99	.66	738
14	14	D	16.0	24.00	9.0	11.30	1.32	30	.82	1.22	.82	894
16	16	B	17.0	27.50	10.50	13.00	1.12	30	70	1.03	.70	942
16	16	D	17.0	27.50	10.50	13.00	1.39	30	.89	1.29	.89	1275
18	18	B	18.0	30.00	12.0	14.70	1.17	30	.75	1.08	.75	1266
18	18	D	18.0	30.00	12.0	14.70	1.46	30	.96	1.36	.96	1607
20	20	B	18.0	34.00	13.50	16.40	1.26	30	.80	1.16	.80	1635
20	20	D	18.0	34.00	13.50	16.40	1.57	30	1.03	1.46	1.03	2296
24	20	B	12.00	34.00	13.50	16.40	1.26	30	.89	1.16	.80	1663
24	20	D	12.00	34.00	13.50	16.40	1.57	30	1.16	1.46	1.03	2393
24	24	B	18.00	38.00	15.25	19.30	1.36	30	.89	1.26	.89	2300
24	24	D	18.00	38.00	15.25	19.30	1.75	30	1.16	1.63	1.16	2957
30	24	A	12.00	38.00	15.25	19.30	1.36	30	.88	1.26	.89	2171
30	24	B	12.00	38.00	15.25	19.30	1.36	30	1.03	1.26	.89	2217
30	24	C	12.00	38.00	15.25	19.30	1.75	30	1.20	1.63	1.16	2717
30	24	D	12.00	38.00	15.25	19.30	1.75	30	1.37	1.63	1.16	2811
30	30	A	18.00	48.00	18.00	23.70	1.32	30	.88	1.22	.88	3153
30	30	B	18.00	48.00	18.00	23.70	1.59	30	1.03	1.47	1.03	3687
30	30	C	18.00	48.00	18.00	23.70	1.88	30	1.20	1.74	1.20	4285
30	30	D	18.00	48.00	18.00	23.70	2.17	30	1.37	2.01	1.37	4941
36	30	A	10.00	48.00	18.00	23.70	1.32	30	.99	1.22	.88	3343
36	30	B	10.00	48.00	18.00	23.70	1.59	30	1.15	1.47	1.03	3874
36	30	C	10.00	48.00	18.00	23.70	1.88	30	1.36	1.74	1.20	4486
36	30	D	10.00	48.00	18.00	23.70	2.17	30	1.58	2.01	1.37	5189
36	36	A	18.00	56.00	21.00	28.20	1.50	24	.99	1.39	.99	4949
36	36	B	18.00	56.00	21.00	28.20	1.79	24	1.15	1.66	1.15	5858
36	36	C	18.00	56.00	21.00	28.20	2.13	24	1.36	1.98	1.36	6804
36	36	D	18.00	56.00	21.00	28.20	2.48	24	1.58	2.31	1.58	8082
42	30	A	6.00	48.00	18.00	23.70	1.32	30	1.10	1.22	.88	3368
42	30	B	6.00	48.00	18.00	23.70	1.59	30	1.28	1.47	1.03	3890
42	30	C	6.00	48.00	18.00	23.70	1.88	30	1.54	1.74	1.20	4543
42	30	D	6.00	48.00	18.00	23.70	2.17	30	1.78	2.01	1.37	5241
42	36	A	10.00	56.00	21.00	28.20	1.50	24	1.10	1.39	.99	4904
42	36	B	10.00	56.00	21.00	28.20	1.79	24	1.28	1.66	1.15	5789
42	36	C	10.00	56.00	21.00	28.20	2.13	24	1.54	1.98	1.36	6761
42	36	D	10.00	56.00	21.00	28.20	2.48	24	1.78	2.31	1.58	8025
42	42	A	18.00	66.00	25.00	33.10	1.72	24	1.10	1.60	1.10	7394
42	42	B	18.00	66.00	25.00	33.10	2.05	24	1.28	1.90	1.28	8417
42	42	C	18.00	66.00	25.00	33.10	2.46	24	1.54	2.28	1.54	10377
42	42	D	18.00	66.00	25.00	33.10	2.85	24	1.78	2.64	1.78	12072
48	36	A	2.00	56.00	21.00	28.20	1.50	24	1.26	1.39	.99	4727
48	36	B	2.00	56.00	21.00	28.20	1.79	24	1.42	1.66	1.15	5584
48	36	C	2.00	56.00	21.00	28.20	2.13	24	1.71	1.98	1.36	6494
48	36	D	2.00	56.00	21.00	28.20	2.48	24	1.96	2.31	1.58	7731
48	42	A	10.00	66.00	25.00	33.10	1.72	24	1.26	1.60	1.10	7345
48	42	B	10.00	66.00	25.00	33.10	2.05	24	1.42	1.90	1.28	8338
48	42	C	10.00	66.00	25.00	33.10	2.46	24	1.71	2.28	1.54	10249
48	42	D	10.00	66.00	25.00	33.10	2.85	24	1.96	2.64	1.78	11924
48	48	A	18.00	76.00	28.00	37.60	1.99	24	1.26	1.86	1.26	10200
48	48	B	18.00	76.00	28.00	37.60	2.32	24	1.42	2.15	1.42	12132
48	48	C	18.00	76.00	28.00	37.60	2.78	24	1.71	2.57	1.71	14716
48	48	D	18.00	76.00	28.00	37.60	3.20	24	1.96	2.95	1.96	16965

STANDARD SPECIAL CASTINGS FOR WATER

Y Branches, Type 2



4 to 20 Inches

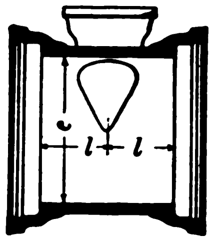


24 to 48 Inches

Table No. 12

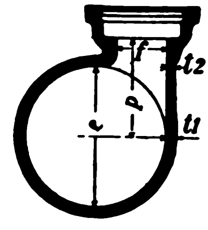
Nominal Diam., Inch.		Class	s	p	v	w	n	r	Thickness Inches		Approx. Weight Pounds
e	f								t_1	t_2	
4	4	D	11.50	10.50	7.18	6.64	2.18	6	.52	.64	103
6	6	D	13.00	13.00	9.27	7.46	3.27	6	.55	.67	181
8	8	D	14.00	16.00	11.85	8.30	3.85	6	.60	.72	291
10	10	D	15.50	18.50	13.94	9.12	4.94	6	.68	.83	434
12	12	D	15,50	21.50	16.54	9.92	4.54	6	.75	.93	632
14	14	B	16.00	24.00	18.62	10.76	4.62	6	.66	.84	690
14	14	D	16.00	24.00	18.62	10.76	4.62	6	.82	1.00	985
16	16	B	17.50	31.00	25.20	11.60	5.70	6	.70	1.03	967
16	16	D	17.50	31.00	25.20	11.60	5.70	6	.89	1.29	1413
18	18	B	18.00	34.00	28.00	12.00	6.00	6	.75	1.12	1358
18	18	D	18.00	34.00	28.00	12.00	6.00	6	.96	1.44	1737
20	20	B	18.75	37.00	30.75	12.50	6.50	6	.80	1.20	1725
20	20	D	18.75	37.00	30.75	12.50	6.50	6	1.03	1.50	2199
24	20	B	18.75	40.00	.....	.....	....	6	.89	.80	2208
24	20	D	18.75	40.00	.....	.....	....	6	1.16	1.03	3087
24	24	B	19.75	42.00	.....	.....	....	6	.89	.89	2600
24	24	D	19.75	42.00	.....	.....	....	6	1.16	1.16	3599
30	24	A	17.00	49.50	.....	.....	....	6	.88	.89	3178
30	24	B	17.00	49.50	.....	.....	....	6	1.03	.89	3874
30	30	A	22.75	52.50	.....	.....	....	6	.88	.88	3519
30	30	B	22.75	52.50	.....	.....	....	6	1.03	1.03	4360
36	30	A	19.75	56.00	.....	.....	....	6	.99	.88	4338
36	30	B	19.75	56.00	.....	.....	....	6	1.15	1.03	4425
36	36	A	24.00	60.00	.....	.....	....	6	.99	.99	4951
36	36	B	24.00	60.00	.....	.....	....	6	1.15	1.15	6509
42	30	A	16.75	63.00	.....	.....	....	6	1.10	.88	5543
42	30	B	16.75	63.00	.....	.....	....	6	1.28	1.03	6782
42	36	A	21.00	66.00	.....	.....	....	6	1.10	.99	6446
42	36	B	21.00	66.00	.....	.....	....	6	1.28	1.15	7895
42	42	A	25.25	69.00	.....	.....	....	6	1.10	1.10	7591
42	42	B	25.25	69.00	.....	.....	....	6	1.28	1.28	9163
48	36	A	18.00	71.00	.....	.....	....	6	1.26	.99	7850
48	36	B	18.00	71.00	.....	.....	....	6	1.42	1.15	9500
48	42	A	22.25	74.00	.....	.....	....	6	1.26	1.10	9116
48	42	B	22 25	74.00	.....	.....	....	6	1.42	1.28	10887
48	48	A	26.50	77.00	.....	.....	....	6	1.26	1.26	10599
48	48	B	26.50	77.00	.....	.....	....	6	1.42	1.42	12554

STANDARD SPECIAL CASTINGS FOR WATER



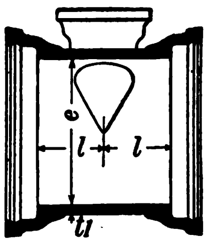
Standard
Blow-off Branches

Table No. 13



Nominal Diameter Inches e	f	Class	l	p	Thickness Inches t_1	t_2	Approx. Weight Pounds	Nominal Diameter Inches e	f	Class	l	p	Thickness Inches t_1	t_2	Approx. Weight Pounds
8	4	D	12	7	.60	.52	227	36	12	A	13	23	.99	.75	1702
10	4	D	12	8	.68	.52	286	36	12	B	13	23	1.15	.75	1972
10	6	D	12	8	.68	.55	300	36	12	C	13	23	1.36	.75	2285
12	4	D	12	10	.75	.52	365	36	12	D	13	23	1.58	.75	2627
12	6	D	12	10	.75	.55	379	42	12	A	15	26	1.10	.75	2432
14	4	B	12	11	.66	.52	400	42	12	B	15	26	1.28	.75	2728
14	4	D	12	11	.82	.52	471	42	12	C	15	26	1.54	.75	3271
14	6	B	12	11	.66	.55	415	42	12	D	15	26	1.78	.75	3768
14	6	D	12	11	.82	.55	486	42	16	A	15	26	1.10	.70	2489
16	4	B	12	12	.70	.52	497	42	16	B	15	26	1.28	.70	2786
16	4	D	12	12	.89	.52	597	42	16	C	15	26	1.54	.89	3365
16	6	B	12	12	.70	.55	513	42	16	D	15	26	1.78	.89	3862
16	6	D	12	12	.89	.55	613	48	12	A	17	30	1.26	.75	3274
18	4	B	12	13	.75	.52	586	48	12	B	17	30	1.42	.75	3690
18	4	D	12	13	.96	.52	704	48	12	C	17	30	1.71	.75	4417
18	6	B	12	13	.75	.55	603	48	12	D	17	30	1.96	.75	5107
18	6	D	12	13	.96	.55	720	48	16	A	17	30	1.26	.70	3337
20	4	B	12	14	.80	.52	687	48	16	B	17	30	1.42	.70	3762
20	4	D	12	14	1.03	.52	850	48	16	C	17	30	1.71	.89	4523
20	6	B	12	14	.80	.55	705	48	16	D	17	30	1.96	.89	5214
20	6	D	12	14	1.03	.55	867	54	12	A	19	33	1.35	.75	4287
24	6	B	12	16	.89	.55	916	54	12	B	19	33	1.55	.75	4945
24	6	D	12	16	1.16	.55	1149	54	12	C	19	33	1.90	.75	5981
24	8	B	12	16	.89	.60	935	54	12	D	19	33	2.23	.75	7002
24	8	D	12	16	1.16	.60	1170	54	16	A	19	33	1.35	.70	4355
30	8	A	13	20	.88	.60	1269	54	16	B	19	33	1.55	.70	5013
30	8	B	13	20	1.03	.60	1382	54	16	C	19	33	1.90	.89	6096
30	8	C	13	20	1.20	.60	1616	54	16	D	19	33	2.23	.89	7126
30	8	D	13	20	1.37	.60	1867	60	12	A	21	36	1.39	.75	5263
30	12	A	13	20	.88	.75	1315	60	12	B	21	36	1.67	.75	6159
30	12	B	13	20	1.03	.75	1426	60	12	C	21	36	2.00	.75	7418
30	12	C	13	20	1.20	.75	1658	60	12	D	21	36	2.38	.75	8798
30	12	D	13	20	1.37	.75	1913	60	16	A	21	36	1.39	.70	5336
36	8	A	13	23	.99	.60	1653	60	16	B	21	36	1.67	.70	6233
36	8	B	13	23	1.15	.60	1922	60	16	C	21	36	2.00	.89	7542
36	8	C	13	23	1.36	.60	2234	60	16	D	21	36	2.38	.89	8927
36	8	D	13	23	1.58	.60	2576								

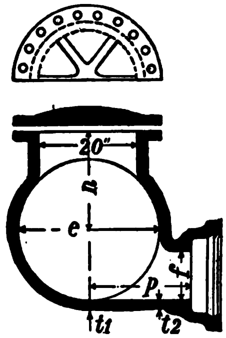
STANDARD SPECIAL CASTINGS FOR WATER



Standard Blow-off Branches with Manhole

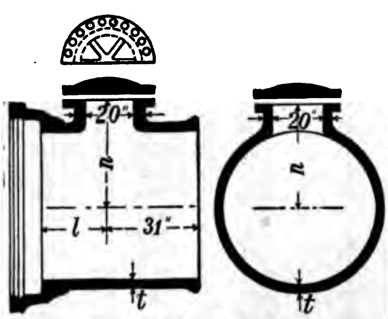
Table No. 14

Approximate Weight of Cap, 290 Pounds



Nom'l Diam. Inches		Class	l	p	n	Thickness Inches		Approximate Weight, Lbs.	Nom'l Diam. Inches		Class	l	p	n	Thickness Inches		Approximate Weight, Lbs.
e	f					t_1	t_2		e	f					t_1	t_2	
30	8	A	17	20	21	.88	.60	1628	48	12	A	17	30	30	1.26	.75	3391
30	8	B	17	20	21	1.03	.60	1758	48	12	B	17	30	30	1.42	.75	3803
30	8	C	17	20	21	1.20	.60	2015	48	12	C	17	30	30	1.71	.75	4497
30	8	D	17	20	21	1.37	.60	2290	48	12	D	17	30	30	1.96	.75	5167
30	12	A	17	20	21	.88	.75	1672	48	16	A	17	30	30	1.26	.70	3454
30	12	B	17	20	21	1.03	.75	1803	48	16	B	17	30	30	1.42	.70	3866
30	12	C	17	20	21	1.20	.75	2057	48	16	C	17	30	30	1.71	.89	4604
30	12	D	17	20	21	1.37	.75	2335	48	16	D	17	30	30	1.96	.89	5274
36	8	A	17	23	24	.99	.60	2045	54	12	A	19	33	33	1.35	.75	4390
36	8	B	17	23	24	1.15	.60	2351	54	12	B	19	33	33	1.55	.75	5032
36	8	C	17	23	24	1.36	.60	2690	54	12	C	19	33	33	1.90	.75	6039
36	8	D	17	23	24	1.58	.60	3071	54	12	D	19	33	33	2.23	.75	7033
36	12	A	17	23	24	.99	.75	2094	54	16	A	19	33	33	1.35	.70	4458
36	12	B	17	23	24	1.15	.75	2395	54	16	B	19	33	33	1.55	.70	5100
36	12	C	17	23	24	1.36	.75	2741	54	16	C	19	33	33	1.90	.89	6154
36	12	D	17	23	24	1.58	.75	3122	54	16	D	19	33	33	2.23	.89	7157
42	12	A	17	26	27	1.10	.75	2726	60	12	A	21	36	36	1.39	.75	5357
42	12	B	17	26	27	1.28	.75	3033	60	12	B	21	36	36	1.67	.75	6230
42	12	C	17	26	27	1.54	.75	3595	60	12	C	21	36	36	2.00	.75	7462
42	12	D	17	26	27	1.78	.75	4109	60	12	D	21	36	36	2.38	.75	8810
42	16	A	17	26	27	1.10	.70	2783	60	16	A	21	36	36	1.39	.70	5429
42	16	B	17	26	27	1.28	.70	3090	60	16	B	21	36	36	1.67	.70	6304
42	16	C	17	26	27	1.54	.89	3689	60	16	C	21	36	36	2.00	.89	7587
42	16	D	17	26	27	1.78	.89	4203	60	16	D	21	36	36	2.38	.89	8939

AMERICAN STANDARD MANHOLE PIPE



Standard Manhole Pipe
Approximate Weight of Cap, 290 Pounds

Table No. 15

Nom'l Diam. Inches	Class	n	t	Weight Pounds	Nom'l Diam. Inches	Class	n	t	Weight Pounds
30	A	21	.88	1536	48	A	30	1.26	3194
30	B	21	1.03	1711	48	B	30	1.42	3610
30	C	21	1.20	1973	48	C	30	1.71	4292
30	D	21	1.37	2245	48	D	30	1.96	4968
36	A	24	99	1953	54	A	33	1.35	4006
36	B	24	1.15	2260	54	B	33	1.55	4598
36	C	24	1.36	2614	54	C	33	1.90	5578
36	D	24	1.58	3012	54	D	33	2.23	6522
42	A	27	1.10	2535	60	A	36	1.39	4750
42	B	27	1.28	2869	60	B	36	1.67	5606
42	C	27	1.54	3445	60	C	36	2.00	6720
42	D	27	1.78	3971	60	D	36	2.38	7959

l—17 inches on 30 inches to 48 inches; 19 inches on 54 inches; 21 inches on 60 inches diameter.

STANDARD SPECIAL CASTINGS FOR WATER

Standard Reducers and Increasers, Type No. 1

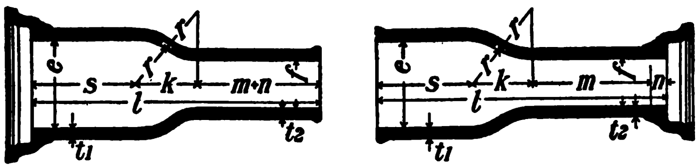


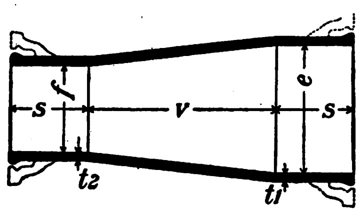
Table No. 16

Diam. Inches		k	m	r	Thickness, Inches		Weights, Pounds	
e	f				t_1	t_2	Large End Bell	Small End Bell
6	4	3.30	14.70	3	.55	.52	99	88
8	4	5.30	12.70	4	.60	.52	131	108
8	6	3.90	14.10	4	.60	.55	149	138
10	4	7.10	10.90	5	.68	.52	164	132
10	6	6.00	12.00	5	.68	.55	181	160
10	8	4.40	13.60	5	.68	.60	205	195
12	6	7.90	10.10	6	.75	.55	225	191
12	8	6.60	11.40	6	.75	.60	246	224
12	10	4.80	13.20	6	.75	.68	271	260

Class D. 6x4 inches to 12x10 inches. On all sizes n = 2 inches.
On all sizes l = 30 inches and s = 10 inches.

AMERICAN STANDARD REDUCERS AND INCREASERS

Type No. 2



6x4 inches to 60x54 inches

Table No. 17

Nominal Diam. Inches		v	Thickness, Inches		Class	Weights, Pounds		
e	f		t_1	t_2		Spigot Ends	Large End Bell	Small End Bell
6	4	18	.55	.52	D	82	104	97
8	4	18	.60	.52	D	104	132	119
8	6	18	.60	.55	D	121	150	143
10	4	18	.68	.52	D	131	162	146
10	6	18	.68	.55	D	150	180	169
10	8	18	.68	.60	D	170	201	198
12	4	18	.75	.52	D	163	201	179
12	6	18	.75	.55	D	181	218	202
12	8	18	.75	.60	D	202	240	231
12	10	18	.75	.68	D	229	267	261
14	6	20	.66	.55	B	194	249	216
14	6	20	.82	.55	D	234	288	256
14	8	20	.66	.60	B	220	275	248
14	8	20	.82	.60	D	260	314	288

On all sizes s = 8 inches.

STANDARD SPECIAL CASTINGS FOR WATER

Standard Reducers and Increasers, Type No. 2

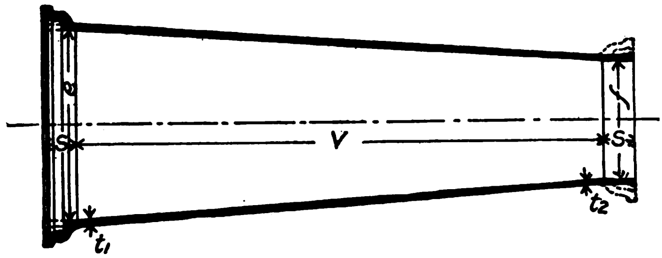
Table No. 17—Continued

Nominal Diam. Inches e	Nominal Diam. Inches f	v	Thickness, Inches t_1	Thickness, Inches t_2	Class	Weights, Pounds Spigot Ends	Weights, Pounds Large End Bell	Weights, Pounds Small End Bell
14	10	20	.66	.68	B	250	305	279
14	10	20	.82	.68	D	290	344	320
14	12	20	.66	.75	B	284	339	321
14	12	20	.82	.75	D	324	378	360
16	6	20	.70	.55	B	226	300	248
16	6	20	.89	.55	D	278	355	300
16	8	20	.70	.60	B	252	326	280
16	8	20	.89	.60	D	304	381	332
16	10	20	.70	.68	B	282	356	312
16	10	20	.89	.68	D	334	410	364
16	12	20	.70	.75	B	317	391	353
16	12	20	.89	.75	D	368	445	405
16	14	20	.70	.66	B	315	389	370
16	14	20	.89	.82	D	407	484	461
18	8	20	.75	.60	B	287	374	315
18	8	20	.96	.60	D	345	438	373
18	10	20	.75	.68	B	317	404	347
18	10	20	.96	.68	D	375	468	405
18	12	20	.75	.75	B	352	438	388
18	12	20	.96	.75	D	410	502	446
18	14	20	.75	.66	B	350	437	406
18	14	20	.96	.82	D	448	541	502
18	16	20	.75	.70	B	383	469	457
18	16	20	.96	.89	D	492	585	569
20	10	26	.80	.68	B	414	516	445
20	10	26	1.03	.68	D	499	615	529
20	12	26	.80	.75	B	455	556	491
20	12	26	1.03	.75	D	539	656	576
20	14	26	.80	.66	B	453	554	508
20	14	26	1.03	.82	D	583	700	638
20	16	26	.80	.70	B	490	592	564
20	16	26	1.03	.89	D	635	751	711
20	18	26	.80	.75	B	581	633	617
20	18	26	1.03	.96	D	683	800	776
24	14	26	.89	.66	B	552	680	607
24	14	26	1.16	.82	D	710	866	764
24	16	26	.89	.70	B	589	717	663
24	16	26	1.16	.89	D	762	917	838
24	18	26	.89	.75	B	630	758	717
24	18	26	1.16	.96	D	810	965	901
24	20	26	.89	.80	B	675	803	776
24	20	26	1.16	1.03	D	871	1027	987
30	18	26	.88	.75	A	710	903	796
30	18	26	1.03	.75	B	791	969	878
30	18	26	1.20	.96	C	956	1166	1048
30	18	26	1.37	.96	D	1054	1305	1146
30	20	26	.88	.80	A	754	947	856

On all sizes s = 8 inches.

STANDARD SPECIAL CASTINGS FOR WATER

Standard Reducers and Increasers, Type No. 2



Long Increaser. 48 to 80 inches x 132 inches v

Table No. 17—Continued

Nominal Diam. Inches		v	Thickness, Inches		Class	Weights, Pounds		
e	f		t_1	t_2		Spigot Ends	Large End Bell	Small End Bell
30	20	26	1.03	.80	B	836	1014	937
30	20	26	1.20	1.03	C	1018	1227	1134
30	20	26	1.37	1.03	D	1115	1366	1232
30	20	66	.88	.80	A	1468	1661	1569
30	20	66	1.03	.80	B	1626	1804	1728
30	20	66	1.20	1.03	C	1981	2190	2098
30	20	66	1.37	1.03	D	2172	2423	2289
30	24	26	.88	.89	A	854	1047	981
30	24	26	1.03	.89	B	935	1113	1063
30	24	26	1.20	1.16	C	1144	1354	1300
30	24	26	1.37	1.16	D	1242	1493	1398
30	24	66	.88	.89	A	1661	1921	1869
30	24	66	1.03	.89	B	1820	1998	1946
30	24	66	1.20	1.16	C	2228	2438	2384
30	24	66	1.37	1.16	D	2419	2670	2575
36	20	32	.99	.80	A	1039	1286	1141
36	20	32	1.15	.80	B	1170	1450	1272
36	20	32	1.36	1.03	C	1417	1739	1534
36	20	32	1.58	1.03	D	1589	1951	1705
36	20	66	.99	.80	A	1771	2018	1872
36	20	66	1.15	.80	B	1994	2274	2095
36	20	66	1 36	1.03	C	2416	2738	2533
36	20	66	1.58	1.03	D	2710	3072	2827
36	24	32	.99	.89	A	1153	1339	1280
36	24	32	1.15	.89	B	1283	1564	1411
36	24	32	1.36	1.16	C	1562	1884	1718
36	24	32	1.58	1.16	D	1734	2096	1890
36	24	66	.99	.89	A	1964	2211	2091
36	24	66	1.15	.89	B	2188	2468	2314
36	24	66	1.36	1.16	C	2664	2985	2820
36	24	66	1 58	1.16	D	2957	3319	3113
36	30	32	.99	.88	A	1213	1490	1436
36	30	32	1.15	1.03	B	1467	1747	1645
36	30	32	1.36	1.20	C	1730	2051	1939
36	30	32	1.58	1.37	D	2013	2375	2264

On all sizes s=8 inches.

STANDARD SPECIAL CASTINGS FOR WATER

Standard Reducers and Increasers, Type No. 2

Table No. 17—Continued

Nominal Diam. Inches		v	Thickness, Inches		Class	Weights, Pounds		
e	f		t_1	t_2		Spigot Ends	Large End Bell	Small End Bell
36	30	66	.99	.89	A	2119	2366	2312
36	30	66	1.15	1.03	B	2502	2783	2680
36	30	66	1.36	1.20	C	2950	3271	3159
36	30	66	1.58	1.37	D	3434	3796	3684
42	20	32	1.10	.80	A	1262	1602	1364
42	20	32	1.28	.80	B	1413	1768	1515
42	20	32	1.54	1.03	C	1753	2168	1869
42	20	32	1.78	1.03	D	1975	2445	2092
42	20	66	1.10	.80	A	2152	2491	2254
42	20	66	1.28	.80	B	2410	2764	2511
42	20	66	1.54	1.03	C	2989	3405	3106
42	20	66	1.78	1.03	D	3369	3839	3486
42	24	32	1.10	.89	A	1376	1715	1504
42	24	32	1.28	.89	B	1527	1881	1654
42	24	32	1.54	1.16	C	1898	2313	2053
42	24	32	1.78	1.16	D	2120	2590	2276
42	24	66	1.10	.89	A	2346	2685	2472
42	24	66	1.28	.89	B	2603	2958	2730
42	24	66	1.54	1.16	C	3237	3652	3392
42	24	66	1.78	1.16	D	3616	4086	3772
42	30	32	1.10	.88	A	1467	1806	1660
42	30	32	1.28	1.03	B	1711	2065	1889
42	30	32	1.54	1.20	C	2065	2480	2275
42	30	32	1.78	1.37	D	2399	2869	2650
42	30	66	1.10	.88	A	2500	2839	2693
42	30	66	1.28	1.03	B	2917	3271	3095
42	30	66	1.54	1.20	C	3523	3938	3732
42	30	66	1.78	1.37	D	4093	4563	4344
42	36	32	1.10	.99	A	1645	1984	1891
42	36	32	1.28	1.15	B	1926	2281	2207
42	36	32	1.54	1.36	C	2320	2735	2642
42	36	32	1.78	1.58	D	2714	3184	3076
42	36	66	1.10	.99	A	2803	3143	3050
42	36	66	1.28	1.15	B	3285	3639	3565
42	36	66	1.54	1.36	C	3958	4373	4279
42	36	66	1.78	1.58	D	4631	5101	4993
48	30	66	1.26	.88	A	2975	3381	3168
48	30	66	1.42	1.03	B	3428	3883	3606
48	30	66	1.71	1.20	C	4092	4641	4801
48	30	66	1.96	1.37	D	4762	5388	5013
48	30	132	1.26	.88	A	5363	5769	5556
48	30	132	1.42	1.03	B	6180	6635	6359
48	30	132	1.71	1.20	C	7379	7928	7588
48	30	132	1.96	1.37	D	8588	9214	8839
48	36	66	1.26	.99	A	3278	3684	3525
48	36	66	1.42	1.15	B	3796	4252	4077
48	36	66	1.71	1.36	C	4527	5076	4849

On all sizes s=8 inches.

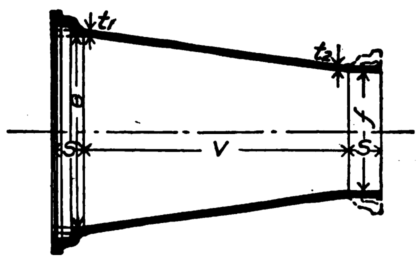
STANDARD SPECIAL CASTINGS FOR WATER
Standard Reducers and Increasers, Type No. 2
Table No. 17—Continued

Nominal Diam. Inches		v	Thickness, Inches		Class	Weights, Pounds		
e	f		t_1	t_2		Spigot Ends	Large End Bell	Small End Bell
48	36	66	1.96	1.58	D	5300	5925	5662
48	36	132	1.26	.99	A	5909	6316	6156
48	36	132	1.42	1.15	B	6844	7299	7125
48	36	132	1.71	1.36	C	8164	8713	8485
48	36	132	1.96	1.58	D	9558	10184	9920
48	42	66	1.26	1.10	A	3659	4066	3998
48	42	66	1.42	1.28	B	4212	4667	4564
48	42	66	1.71	1.54	C	5100	5649	5516
48	42	66	1.96	1.78	D	5959	6585	6420
48	42	132	1.26	1.10	A	6597	7003	6936
48	42	132	1.42	1.28	B	7594	8049	7948
48	42	132	1.71	1.54	C	9197	9746	9612
48	42	132	1.96	1.78	D	10747	11373	11217
54	36	66	1.35	.99	A	3722	4228	3969
54	36	66	1.55	1.15	B	4330	4925	4610
54	36	66	1.90	1.36	C	5259	5953	5580
54	36	66	2.23	1.58	D	6181	6995	6543
54	36	132	1.35	.99	A	6710	7216	6957
54	36	132	1.55	1.15	B	7806	8401	8087
54	36	132	1.90	1.36	C	9484	10178	9805
54	36	132	2.23	1.58	D	11148	11962	11510
54	42	66	1.35	1.10	A	4103	4609	4442
54	42	66	1.55	1.28	B	4745	5340	5100
54	42	66	1.90	1.54	C	5832	6526	6247
54	42	66	2.23	1.78	D	6841	7655	7310
54	42	132	1.35	1.10	A	7398	7903	7737
54	42	132	1.55	1.28	B	8556	9151	8910
54	42	132	1.90	1.54	C	10517	11211	10932
54	42	132	2 23	1.78	D	12338	13152	12807
54	48	66	1.35	1.26	A	4578	5083	4984
54	48	66	1.55	1.42	B	5256	5851	5711
54	48	66	1.90	1.71	C	6401	7095	6950
54	48	66	2.23	1.96	D	7512	8326	8137
54	48	132	1.35	1.26	A	8253	8759	8660
54	48	132	1.55	1.42	B	9478	10073	9933
54	48	132	1.90	1.71	C	11544	12239	12093
54	48	132	2.23	1 96	D	13550	14364	14175
60	36	66	1.39	.99	A	4096	4711	4342
60	36	66	1.67	1.15	B	4906	5576	5186
60	36	66	2.00	1.36	C	5867	6692	6189
60	36	66	2.38	1.58	D	6960	7934	7322
60	36	132	1.39	.99	A	7384	7999	7631
60	36	132	1.67	1.15	B	8846	9516	9126
60	36	132	2.00	1.36	C	10581	11405	10902
60	36	132	2.38	1.58	D	12554	13527	12916
60	42	66	1.39	1.10	A	4477	5092	4816
60	42	66	1.67	1.28	B	5321	5991	5676

On all sizes s=8 inches.

STANDARD SPECIAL CASTINGS FOR WATER

Standard Reducers and Increasers, Type No. 2



Short Increaser, 48 to 30 x 66 inches v

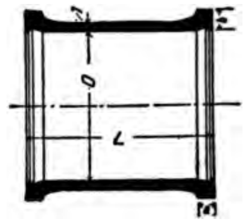
Table No. 17—Continued

Nominal Diam. Inches		v	Thickness, Inches		Class	Weights, Pounds		
e	f		t_1	t_2		Spigot Ends	Large End Bell	Small End Bell
60	42	66	2.00	1.54	C	6440	7264	6855
60	42	66	2.38	1.78	D	7619	8593	8089
60	42	132	1.39	1.10	A	8072	8687	8411
60	42	132	1.67	1.28	B	9595	10265	9950
60	42	132	2.00	1.54	C	11614	12439	12030
60	42	132	2.38	1.78	D	13743	14716	14213
60	48	66	1.39	1.26	A	4957	5572	5363
60	48	66	1.67	1.42	B	5832	6502	6287
60	48	66	2.00	1.71	C	7006	7830	7555
60	48	66	2.38	1.96	D	8285	9259	8910
60	48	132	1.39	1.26	A	8938	9552	9344
60	48	132	1.67	1.42	B	10517	11187	10972
60	48	132	2.00	1.71	C	12634	13458	13183
60	48	132	2.38	1.96	D	14943	15917	15568
60	54	66	1.39	1.35	A	5404	6019	5910
60	54	66	1.67	1.55	B	6348	7018	6961
60	54	66	2.00	1.90	C	7750	8574	8444
60	54	66	2.38	2.23	D	9178	10152	9992
60	54	132	1.39	1.35	A	9745	10360	10251
60	54	132	1.67	1.55	B	11462	12132	12075
60	54	132	2.00	1.90	C	13979	14808	14678
60	54	132	2.38	2.23	D	16557	17530	17371

On all sizes s=8 inches.

STANDARD SPECIAL CASTINGS FOR WATER

Standard Sleeves



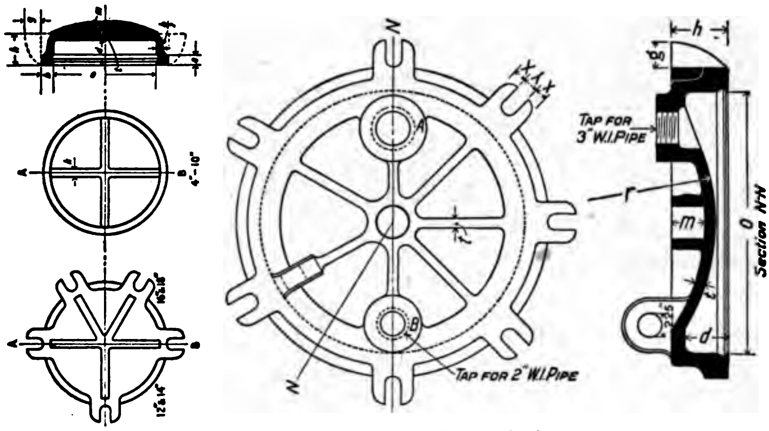
For dimensions a and b see Table No. 1

Table No. 18

Nom'l Diam. Inches	Class	D	L	T	Approx. Weight Pounds	Nom'l Diam. Inches	Class	D	L	T	Approx. Weight Pounds
4	D	5.80	10	.65	47	36	B	39.40	15	1.40	943
4	D	5.80	15	.65	61	36	C	39.80	15	1.60	1077
6	D	7.90	10	.70	68	36	D	40.20	15	1.80	1217
6	D	7.90	15	.70	87	36	A	39.00	24	1.25	1202
8	D	10.10	12	.75	104	36	B	39.40	24	1.40	1362
8	D	10.10	15	.75	119	36	C	39.80	24	1.60	1563
10	D	12.20	12	.80	123	36	D	40.20	24	1.80	1772
10	D	12.20	18	.80	176	42	A	45.30	15	1.40	1097
12	D	14.30	14	.85	174	42	B	45.60	15	1.50	1184
12	D	14.30	18	.85	223	42	C	46.20	15	1.75	1381
14	B	16.20	15	.85	220	42	D	46.70	15	1.95	1561
14	B	16.20	18	.85	249	42	A	45.30	24	1.40	1577
14	D	16.50	15	.90	240	42	B	45.60	24	1.50	1702
14	D	16.50	18	.90	280	42	C	46.20	24	1.75	1997
16	B	18.50	15	.90	274	42	D	46.70	24	1.95	2262
16	B	18.50	24	.90	391	48	A	51.60	15	1.50	1337
16	D	18.90	15	1.00	305	48	B	51.90	15	1.65	1481
16	D	18.90	24	1.00	443	48	C	52.50	15	1.95	1752
18	B	20.60	15	.95	321	48	D	53.10	15	2.20	1986
18	B	20.60	24	.95	462	48	A	51.60	24	1.50	1922
18	D	21.00	15	1.05	360	48	B	51.90	24	1.65	2129
18	D	21.00	24	1.05	518	48	C	52.50	24	1.95	2532
20	B	22.70	15	1.00	374	48	D	53.10	24	2.20	2879
20	B	22.70	24	1.00	532	54	A	57.70	15	1.60	1612
20	D	23.10	15	1.15	440	54	B	58.20	15	1.80	1835
20	D	23.10	24	1.15	625	54	C	58.90	15	2.15	2156
24	B	26.90	15	1.05	477	54	D	59.50	15	2.45	2450
24	B	26.90	24	1.05	680	54	A	57.70	24	1.60	2316
24	D	27.40	15	1.25	583	54	B	58.20	24	1.80	2634
24	D	27.40	24	1.25	821	54	C	58.90	24	2.15	3126
30	A	32.80	15	1.15	648	54	D	59.50	24	2.45	3571
30	B	33.10	15	1.15	652	60	A	63.90	15	1.70	1906
30	C	33.50	15	1.32	760	60	B	64.50	15	1.90	2127
30	D	33.80	15	1.50	876	60	C	65.30	15	2.25	2491
30	A	32.80	24	1.15	943	60	D	65.90	15	2.60	2895
30	B	33.10	24	1.15	949	60	A	63.90	24	1.70	2731
30	C	33.50	24	1.32	1088	60	B	64.50	24	1.90	3058
30	D	33.80	24	1.50	1262	60	C	65.30	24	2.25	3601
36	A	39.00	15	1.25	833	60	D	65.90	24	2.60	4231

STANDARD SPECIAL CASTINGS FOR WATER

Standard Caps. Table No. 19



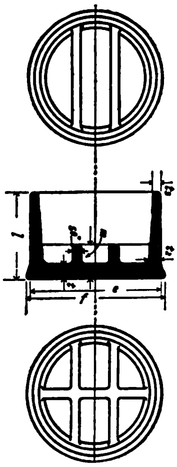
Bosses A and B cast on only when so ordered

Bosses A and B cast on only when so ordered

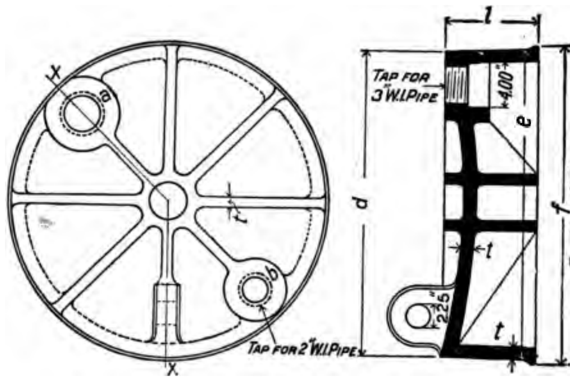
Nominal Diam. Inches	Class	d	o	l	t	m	k	r	Approx. Weight Pounds
4	D	4.00	5.70	....	.60	....	....	.....	26
6	D	4.00	7.80	....	.65	....	....	.....	40
8	D	4.00	10.00	....	.75	....	....	.....	59
10	D	4.00	12.10	....	.75	1.50	.75	16.20	81
12	D	4.00	14.20	....	.75	1.75	.75	18.70	104
14	B	4.00	16.10	....	.90	1.90	.75	22.40	140
14	D	4.00	16.45	....	.90	1.90	.75	22.40	149
16	B	4.00	18.40	....	1.00	2.00	.75	27.00	183
16	D	4.00	18.80	....	1.00	2.00	.75	27.00	198
18	B	4.00	20.50	....	1.00	2.00	1.00	32 00	226
18	D	4.00	20.92	....	1.00	2.00	1.00	32.90	242
20	B	4.00	22.60	....	1.00	3.00	1.00	18.20	278
20	D	4.00	23.06	.....	1.00	3.00	1.00	18.20	308
24	B	4.00	26.80	2.50	1.05	3.50	1.00	23.50	392
24	D	4.00	27.32	2.50	1.05	3.50	1.00	23.50	442
30	A	4.50	32.74	2.62	1.15	3.50	1.15	34.80	589
30	B	4.50	33.00	2.62	1.15	3.50	1.15	34.80	596
30	C	4.50	33.40	2.62	1.15	3.50	1.15	34.80	647
30	D	4.50	33.74	2.62	1.15	3.50	1.15	34.80	704
36	A	4.50	38.96	3.12	1.25	4.00	1.25	44.00	849
36	B	4.50	39.30	3.12	1.30	3.95	1.25	44.00	918
36	C	4.50	39.70	3.12	1.35	3.90	1.25	44.00	998
36	D	4.50	40.16	3.12	1.40	3.85	1.25	44.00	1084
42	A	5.00	45.20	3.37	1.40	4.00	1.40	63.50	1300
42	B	5.00	45.50	3.37	1.50	3.90	1.40	63.50	1388
42	C	5.00	46.10	3.37	1.60	3.80	1.40	63.50	1539
42	D	5.00	46.58	3.37	1.70	3.70	1.40	63.50	1679
48	A	5.00	51.50	3.62	1.70	4.00	1.50	76.50	1772
48	B	5.00	51.80	3.62	1.90	3.80	1.50	76.50	1943
48	C	5.00	52.40	3.62	2.00	3.70	1.50	76.50	2144
48	D	5.00	52.98	3.62	2.10	3.60	1.50	76.50	2341
54	A	5.50	57.66	3.87	1.90	4.50	1.50	82.00	2329
54	B	5.50	58.10	3.87	2.00	4.40	1.50	82.00	2519
54	C	5.50	58.80	3.87	2.10	4.30	1.50	82.00	2770
54	D	5.50	59.40	3.87	2.20	4.20	1.50	82.00	3009
60	A	5.50	63.80	4.12	2.00	4.50	1.50	99.00	2868
60	B	5.50	64.40	4.12	2.10	4.40	1.50	99.00	3082
60	C	5.50	65.20	4.12	2.20	4.30	1.50	99.00	3388
60	D	5.50	65.82	4.12	2.30	4.20	1.50	99.00	3687

STANDARD SPECIAL CASTINGS FOR WATER

Standard Plugs



4 to 20 inches



24 to 60 inches

Bosses a and b cast on only when so ordered

Table No. 20

Nominal Diam. Inches	Class	e	f	d	l	m	Thickness, Inches			Number of Ribs	Approx. Weight Pounds
							t	t_2	t_3		
4	D	4.90	5.28	.....	5.50	....	.50	.40	.20	..	8
6	D	7.00	7.38	.....	5.50	....	.60	.40	.20	..	14
8	D	9.15	9.65	.....	5.50	2.0	.60	.40	.20	2	24
10	D	11.20	11.70	.....	6.00	2.0	.70	.50	.20	2	38
12	D	13.30	13.80	.....	6.00	2.0	.75	.50	.20	2	50
14	B	15.30	15.80	.....	6.00	2.0	70	.50	.20	2	63
14	D	15.65	16.15	.....	6.00	2.0	.75	.50	.20	2	65
16	B	17.40	17.90	.....	6.50	2.0	.70	.50	.30	3	90
16	D	17.80	18.30	.....	6.50	2.0	.80	.60	.30	3	96
18	B	19.50	20.00	.....	6.50	2.50	.75	.60	.30	3	111
18	D	19.92	20.42	.....	6.50	2.50	.85	.60	.30	3	121
20	B	21.60	22.10	.....	6.50	2.75	.85	.60	.30	3	151
20	D	22.06	22.56	.....	6.50	2.75	1.00	.60	.30	3	156
24	B	25.92	26.30	25.68	8.0	....	.89	...	...	4	375
24	D	26.44	26.82	26.20	8.0	....	1.16	...	...	4	472
30	A	31.86	32.24	31.62	8.0	....	.88	...	...	4	481
30	B	32.12	32.50	31.88	8.0	....	1.03	...	...	4	556
30	C	32.52	32.90	32.28	8.0	....	1.20	...	...	4	641
30	D	32.86	33.24	32.62	8.0	....	1.37	...	...	4	723
36	A	38.08	38.46	37.84	8.0	....	.99	...	...	4	682
36	B	38.42	38.80	38.18	8.0	....	1.15	...	...	4	786
36	C	38.82	39.20	38.58	8.0	....	1.36	...	...	4	914
36	D	39.28	39.66	39.04	8.0	....	1.58	...	...	4	1050
42	A	44.32	44.70	44.08	9.0	....	1.10	...	...	4	991
42	B	44.62	45.00	44.38	9.0	....	1.28	...	...	4	1138
42	C	45.22	45.60	44.98	9.0	....	1.54	...	...	4	1353
42	D	45.70	46.08	45.46	9.0	....	1.78	...	...	4	1551
48	A	50.62	51.00	50.38	9.0	....	1.26	...	...	4	1349
48	B	50.92	51.30	50.68	9.0	....	1.42	...	...	4	1506
48	C	51.52	51.90	51.28	9.0	....	1.71	...	...	4	1800
48	D	52.10	52.48	51.86	9.0	....	1.96	...	...	4	2047
54	A	56.78	57.16	56.54	9.0	....	1.35	...	...	4	1697
54	B	57.22	57.60	56.98	9.0	....	1.55	...	...	4	1945
54	C	57.92	58.30	57.68	9.0	....	1.90	...	...	4	2356
54	D	58.52	58.90	58.28	9.0	....	2.23	...	...	4	2733
60	A	62.92	63.30	62.68	9.0	....	1.39	...	...	4	2045
60	B	63.52	63.90	63.28	9.0	....	1.67	...	...	4	2434
60	C	64.32	64.70	64.08	9.0	....	2.00	...	...	4	2904
60	D	64.94	65.32	64.70	9.0	....	2.38	...	...	4	3397

Spiral Riveted Pressure Pipe

American Spiral Pipe Works

Inside Diameter Inches	Thickness U. S. Standard Gauge	Approximate Weight in Pounds per Foot Asphalted	Approximate Bursting Strength Pounds per Square Inch
3	20 *	1.9	1500
	18 * *	2.3	2000
4	20	2.4	1125
	18 *	3.0	1500
	16 * *	3.7	1875
5	20	2.9	900
	18 *	3.7	1200
	16 * *	4.5	1500
6	18	4.3	1000
	16 *	5.3	1250
	14 * *	6.6	1560
	12 * * *	9.2	2170
7	18	5.1	860
	16 *	6.2	1070
	14 * *	7.7	1340
	12 * * *	10.7	1860
8	18	5.8	750
	16 *	7.1	935
	14 * *	8.8	1170
	12 * * *	12.3	1640
9	16 *	8.0	835
	14 * *	9.9	1045
	12 * * *	13.9	1460
10	16 *	8.8	750
	14 * *	11.0	935
	12 * * *	15.3	1310
11	16 *	9.7	680
	14 * *	12.0	850
	12 * * *	16.6	1200
12	16 *	10.6	625
	14 * *	13.0	780
	12 * * *	18.2	1080
	10	22.5	1410

* denotes Standard thickness for Spiral Riveted Pipe.
* * extra heavy.
* * * double extra heavy.

Spiral Riveted Pressure Pipe

CONTINUED

Inside Diameter Inches	Thickness U. S. Standard Gauge	Approximate Weight in Pounds per Foot Asphalted	Approximate Bursting Strength Pounds per Square Inch
13	16 *	11.4	575
	14 * *	14.1	720
	12 * * *	19.7	1010
	10	24.5	1295
14	16	12.9	535
	14 *	15.9	670
	12 * *	22.2	940
	10 * * *	27.6	1210
15	14 *	17.0	625
	12 * *	23.7	875
	10 * * *	29.6	1125
16	14 *	18.1	585
	12 * *	25.2	820
	10 * * *	31.5	1050
	8	38.1	1290
	6	44.7	1520
	3	51.6	1880
18	14 *	19.9	520
	12 * *	27.6	730
	10 * * *	34.5	940
	8	41.6	1140
	6	49.0	1360
	3	59.2	1660
20	14 *	22.1	470
	12 * *	30.6	660
	10 * * *	38.3	840
	8	46 2	1030
	6	54.1	1220
	3	65.6	1500
22	14	24.4	425
	12 *	33.7	595
	10 * *	42.2	765
	8 * * *	50.8	940
	6	59 5	1108
	3	72.2	1364
24	14	26.4	390
	12 *	36.5	540
	10 * *	45.7	705
	8 * * *	55.2	820
	6	64.6	1015
	3	78.4	1250

* denotes Standard thickness for Spiral Riveted Pipe.

** extra heavy.

*** double extra heavy.

Spiral Riveted Pressure Pipe

CONTINUED

Inside Diameter Inches	Thickness U. S. Standard Gauge	Approximate Weight in Pounds per Foot Asphalted	Approximate Bursting Strength, Pounds per Square Inch
26	12 *	39.5	505
	10 * *	49.5	650
	8 * * *	59.8	795
	6	70.0	935
	3	84.9	1154
28	12	42.1	470
	10 *	51.7	605
	8 * *	63.6	735
	6 * * *	76.6	870
	3	90.4	1071
30	12	45.3	435
	10 *	56.8	560
	8 * *	68.7	685
	6 * * *	80.5	810
	3	97.7	1000
32	12	49.1	410
	10 *	61.6	525
	8 * *	74.3	645
	6 * * *	87.1	760
	3	105.8	940
34	12	52.1	380
	10 *	65.4	490
	8 * *	78.8	600
	6 * * *	93.6	715
	3	112.3	880
36	12	55.1	365
	10 *	69.1	470
	8 * *	83.4	570
	6 * * *	97.8	680
	3	118.8	830
40	12	61.1	330
	10 *	76.7	420
	8 * *	92.4	515
	6 * * *	108.5	610
	3	131.8	750

* Denotes Standard thickness for Spiral Riveted Pipe.

* * extra heavy.

* * * double extra heavy.

"NATIONAL" Standard Pipe—Black and Galvanized

All Weights and Dimensions are Nominal

Size	Diameters		Thickness	Weight per foot		Threads per inch	Couplings		
	External	Internal		Plain ends	Threads and couplings		Diameter	Length	Weight
1/8	.405	.269	068	.244	.245	27	.562	7/8	.029
1/4	540	.364	.088	.424	.425	18	.685	1	.043
3/8	675	.493	.091	.567	.568	18	.848	1 1/8	.070
1/2	840	.622	109	.850	.852	14	1.024	1 3/8	116
3/4	1.050	.824	.113	1.130	1 134	14	1 281	1 5/8	.209
1	1.315	1.049	133	1.678	1.684	11 1/2	1.576	1 7/8	343
1 1/4	1.660	1.380	.140	2.272	2.281	11 1/2	1.950	2 1/8	535
1 1/2	1.900	1.610	.145	2.717	2.731	11 1/2	2.218	2 3/8	743
2	2.375	2.067	154	3.652	3.678	11 1/2	2.760	2 5/8	1.208
2 1/2	2.875	2.469	.203	5 793	5.819	8	3.276	2 7/8	1.720
3	3.500	3.068	.216	7.575	7.616	8	3.948	3 1/8	2.498
3 1/2	4.000	3.548	226	9 109	9.202	8	4.591	3 5/8	4.241
4	4.500	4.026	.237	10.790	10.889	8	5.091	3 5/8	4.741
4 1/2	5.000	4.506	.247	12.538	12.642	8	5.591	3 5/8	5.241
5	5.563	5.047	.258	14.617	14.810	8	6.296	4 1/8	8.091
6	6.625	6.065	.280	18.974	19.185	8	7.358	4 1/8	9.554
7	7.625	7.023	.301	23.544	23.769	8	8.358	4 1/8	10.932
8	8.625	8.071	.277	24.696	25.000	8	9.358	4 5/8	13.905
8	8.625	7.981	.322	28.554	28.809	8	9.358	4 5/8	13.905
9	9.625	8.941	.342	33.907	34.188	8	10.358	5 1/8	17.236
10	10.750	10.192	.279	31.201	32.000	8	11.721	6 1/8	29.877
10	10.750	10.136	.307	34.240	35.000	8	11.721	6 1/8	29.877
10	10.750	10.020	.365	40.483	41.132	8	11.721	6 1/8	29.877
11	11.750	11.000	.375	45.557	46 247	8	12.721	6 1/8	32.550
12	12.750	12.090	.330	43.773	45.000	8	13.958	6 1/8	43.098
12	12.750	12.000	.375	49.562	50.706	8	13.958	6 1/8	43.098
13	14.000	13.250	.375	54.568	55.824	8	15.208	6 1/8	47.152
14	15.000	14.250	.375	58.573	60.375	8	16.446	6 1/8	59.493
15	16.000	15.250	.375	62.579	64.500	8	17.446	6 1/8	63.294

The permissible variation in weight is 5 per cent above and 5 per cent below.

Furnished with threads and couplings and in random lengths unless otherwise ordered.

Taper of threads is 3/4 inch diameter per foot length for all sizes.

The weight per foot of pipe with threads and couplings is based on a length of 20 feet, including the coupling, but shipping lengths of small sizes will usually average less than 20 feet.

All weights given in pounds. All dimensions given in inches.

On sizes made in more than one weight, weight desired must be specified.

"NATIONAL" Extra Strong Pipe—Black and Galvanized
All Weights and Dimensions are Nominal

Size	Diameters		Thickness	Weight per foot plain ends
	External	Internal		
1/8	.405	.215	.095	.314
1/4	.540	.302	.119	535
3/8	.675	.423	.126	.738
1/2	.840	.546	147	1 087
3/4	1.050	.742	.154	1.473
1	1.315	.957	.179	2.171
1 1/4	1.660	1.278	.191	2.996
1 1/2	1.900	1.500	.200	3.631
2	2.375	1.939	.218	5.022
2 1/2	2.875	2.323	.276	7.661
3	3.500	2.900	.300	10.252
3 1/2	4.000	3.364	.318	12.505
4	4.500	3.826	.337	14.983
4 1/2	5.000	4.290	.355	17.611
5	5.563	4.813	.375	20.778
6	6.625	5.761	.432	28.573
7	7.625	6.625	.500	38.048
8	8.625	7.625	.500	43.388
9	9.625	8.625	.500	48.728
10	10.750	9.750	.500	54.735
11	11.750	10.750	.500	60 075
12	12.750	11.750	.500	65.415
13	14.000	13.000	.500	72.091
14	15.000	14.000	.500	77.431
15	16.000	15.000	.500	82.771

The permissible variation in weight is 5 per cent above and 5 per cent below.

"NATIONAL" Double Extra Strong Pipe—Black and Galvanized
All Weights and Dimensions are Nominal

Size	Diameters		Thickness	Weight per foot plain ends
	External	Internal		
1/2	.840	.252	.294	1.714
3/4	1.050	.434	.308	2.440
1	1.315	.599	.358	3.659
1 1/4	1.660	.896	.382	5.214
1 1/2	1.900	1.100	.400	6.408
2	2.375	1.503	.436	9.029
2 1/2	2.875	1.771	.552	13.695
3	3.500	2.300	.600	18.583
3 1/2	4.000	2.728	.636	22.850
4	4.500	3.152	.674	27.541
4 1/2	5.000	3.580	.710	32.530
5	5.563	4.063	.750	38.552
6	6.625	4.897	.864	53.160
7	7.625	5.875	.875	63.079
8	8 625	6 875	.875	72.424

**The permissible variation in weight is 10 per cent above and 10 per cent below.
Furnished with plain ends and in random lengths unless otherwise ordered.
All weights given in pounds. All dimensions given in inches.**

"NATIONAL" Matheson Joint Pipe

All Weights and Dimensions are Nominal

External diameter	Thickness	Outside diameter of reinforcing ring — D	Length of joint — L	Weight per foot		Weight of lead per joint
				Plain ends	Complete	
2.00	.095	2.966	2.16	1.932	1.952	1.00
3.00	.109	4.034	2.26	3.365	3.392	1.75
4.00	.128	5.236	2.32	5.293	5.339	2.75
5.00	.134	6.268	2.38	6.963	7.019	3.50
6.00	.140	7.446	2.50	8.762	8.872	4.75
7.00	.149	8.484	2.58	10.902	11.028	5.50
8.00	.158	9.646	2.73	13.233	13.405	6.75
8.00	.185	9.700	2.78	15.441	15.614	6.75
9.00	.167	10.684	2.73	15.754	15.945	8.25
9.00	.196	10.742	2.90	18.429	18.621	8.50
9.00	.250	10.850	3.07	23.362	23.557	9.00
10.00	.175	11.846	2.82	18.363	18.610	9.50
10.00	.208	11.912	2.85	21.752	22.001	9.75
10.00	.270	12.036	3.06	28.057	28.309	10.00
11.00	.185	12.886	2.91	21.368	21.638	11.00
11.00	.220	12.956	2.93	25.329	25.600	11.00
11.00	.290	13.096	3.17	33.171	33.445	12.50
12.00	.194	14.048	3.00	24.461	24.880	13.25
12.00	.244	14.148	3.40	30.635	31.057	14.25
12.00	.310	14.280	3.76	38.703	39.129	16.50
13.00	.202	15.084	3.07	27.610	28.060	15.25
13.00	.247	15.174	3.40	33.642	34.095	15.50
13.00	.310	15.300	3.76	42.014	42.472	18.00
14.00	.210	16.370	3.15	30.928	31.536	17.25
14.00	.250	16.450	3.53	36.713	37.324	19.25
14.00	.310	16.570	3.84	45.325	45.941	20.75
15.00	.222	17.394	3.24	35.038	35.686	19.25
15.00	.260	17.470	3.53	40.930	41.581	20.25
15.00	.320	17.590	3.84	50.171	50.826	22.25
16.00	.234	18.438	3.32	39.401	40.089	22.00
16.00	.270	18.510	3.62	45.359	46.050	23.25
16.00	.330	18.630	3.75	55.228	55.923	24.25
17.00	.240	19.470	3.41	42.959	43.687	23.75
18.00	.245	20.730	3.50	46.458	47.384	25.75
18.00	.310	20.860	3.87	58.568	59.501	28.50
19.00	.259	21.778	3.57	51.840	52.815	29.00
20.00	.272	22.804	3.64	57.309	58.332	31.00
20.00	.375	23.010	4.17	78.599	79.631	35.50
22.00	.301	24.882	4.06	69.756	71.098	40.25
22.00	.400	25.080	4.65	92.276	93.629	45.50
24.00	.330	26.980	4.26	83.423	84.882	48.00
26.00	.362	29.064	4.40	99.122	100.697	55.25
28.00	.396	31.672	4.58	116.746	119.021	65.00
30.00	.432	33.764	4.75	136.421	138.851	75.00

The permissible variation in weight is 5 per cent above and 5 per cent below.

Furnished in random lengths unless otherwise ordered. The weight per foot complete is based on a length of 18 feet of pipe, but shipping lengths of small sizes will usually average less than 18 feet. On sizes made in more than one weight, weight desired must be specified. Column marked weight complete includes the ring but not the lead. Pipe furnished black, galvanized, or dipped. Lead not furnished. All weights given in pounds. All dimensions given in inches.

UNIVERSAL CAST IRON PIPE

Nominal inside diameter	For 100 lbs. pressure			For 130 lbs. pressure			For 175 lbs. pressure			For 250 lbs. pressure			Bolt sizes
	Approx. thickness, inches	Estimated weight, pounds per		Approx. thickness, inches	Estimated weight, pounds per		Approx. thickness, inches	Estimated weight, pounds per		Approx. thickness, inches	Estimated weight, pounds per		
		Foot	6-foot l'ngth		Foot	6-foot l'ngth		Foot	6-foot l'ngth		Foot	6-foot l'ngth	
2	......	....	....	......	....	....	0.35	8½	51	0.39	9½	57	½ × 3½
3	......	....	....	......	....	....	0.37	13	78	0.42	14½	87	½ × 4
4	0.37	18	108	0.40	18¾	112½	0.43	20¼	121½	0.45	21¼	127½	⅝ × 5
5	0.40	24	144	0.425	25	150	0.45	26	156	0.49	29	174	⅝ × 5½
6	0.43	30	180	0.45	31	186	0.47	32	192	0.51	35½	213	¾ × 6
8	0.47	44¼	265½	0.49	46	276	0.525	49½	295½	0.58	53¼	319½	⅞ × 6½
10	0.50	60½	363	0.53	63½	381	0.58	67¾	406½	0.64	74	444	1 × 7½
12	0.53	75½	453	0.57	80½	483	0.62	87	522	0.70	97½	585	1 × 8
14	0.565	94½	567	0.60	99½	597	0.66	107½	645	0.76	124	744	1⅛ × 9
16	0.60	115½	693	0.65	123	738	0.72	134	804	0.83	156	936	1¼ × 9½
20	0.67	166	996	0.73	178	1068	0.82	196	1176	0.94	223	1338	1⅜ × 11

Lengths lay a full six feet. All pipe tested with hydrostatic pressure of 300 pounds per square inch.

FIG. 357*b*. — Universal Cast Iron Pipe.
Standard 6-foot length.

APPENDIX II

STANDARD SPECIFICATIONS FOR HYDRANTS AND VALVES

ADOPTED BY THE AMERICAN WATER WORKS ASSOCIATION, *June* 24, 1913.
REVISED *June* 6, 1916

SPECIFICATIONS FOR HYDRANTS

Size. — The size of hydrant shall be designated by the nominal diameter of the valve opening, which must be at least 4 ins. for hydrants having two 2½-in. hose nozzles; 5 ins. for hydrants having three 2½-in. nozzles; and 6 ins. for hydrants having four 2½-in. nozzles; and shall be classed as one-way, two-way, three-way or four-way, etc., according to the number of 2½-in. hose-outlets for which they are designed.

Area of Water-Way. — The net area of the hydrant at the smallest part, when the valve is wide open, must not be less than 120 per cent that of the valve opening.

Bell Ends or Flange Ends. — All hydrants must be fitted with bell ends to fit standard cast iron pipe; or if flanged, they must be fitted with flanges of the standard dimensions corresponding to the pressure under which they are to be used; connecting pipe or flange from main to hydrant in no case to be less in diameter than the valve opening. (The standards referred to are those adopted, or that may be adopted, by this Association.)

Type. — Hydrants may be of compression or gate type.

Change in Diameter. — Any change in diameter of the water passage through the hydrant must have easy curve, and all outlets must have rounded corners of good radius.

Water Hammer. — Hydrants must be so designed, particularly as regards the pitch of the thread of the operating stem, that, when properly operated, a water hammer will not be caused which will give an increased pressure to exceed the working pressure, when such pressure is over 60 lbs., nor increase the pressure more than 60 lbs., when operated under less working pressure than 60 lbs.

Broken Hydrant. — Valves when shut must remain reasonably tight when upper portion of barrel is broken off.

Friction Loss. — With a 5-ft. hydrant discharging 250 gals. per minute, through each $2\frac{1}{2}$-in. outlet, the total friction loss of the hydrant must not exceed 2 lbs. for two-way, 3 lbs. for three-way, and 4 lbs. for four-way hydrants.

Strapping. — When requested, hydrants must be fitted with 2 lugs, so that the leaded joint underground can be strapped.

Flange Joints above Ground. — When hydrant barrel is made in two sections, the upper flange connection must be at least 2 ins. above the ground line.

Hydrant Body. — The hydrant body must be made of cast iron.

Cast Iron. — All castings shall be made from a superior quality of iron, remelted in cupola or air furnace, tough and even grain, and shall possess a tensile strength of 22,000 lbs. per square inch. The casting must be clean and perfect, without blow or sand holes, or defects of any kind. No plugging or stopping of holes will be allowed.

Specimen Bars. — Specimen bars of the metal used, each being 26 ins. long by 2 ins. wide and 1 in. thick, shall be made without charge, as often as the engineer may direct, and in default of definite instructions, the contractor shall make and test at least one bar from each heat or run of metal. The bars when placed flatwise upon supports, 24 ins. apart, and loaded in the center, shall support a load of 2200 lbs., and show a deflection of not less than 0.35 of an inch, before breaking; or, if preferred, tensile bars shall be made which shall show a breaking point of not less than 22,000 lbs. per square inch. Bars must be cast as nearly as possible to the dimensions without finishing, but corrections may be made by the engineer for variations in width and thickness, and the corrected result must conform to the above requirements.

Wrought Iron. — All wrought iron shall be of the best quality of refined iron of a tensile strength of at least 45,000 lbs. per square inch.

Composition Metals. — All composition or other non-corrodible metals used to be of the best quality, to have a tensile strength of not less than 32,000 lbs. per square inch, with a 5 per cent reduction of area at breaking point.

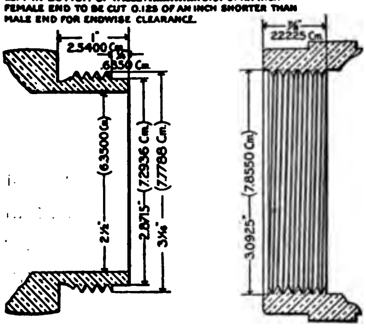
Hose-nipples. — Hose-nipples must be of bronze or suitable non-corrodible metal, either threaded with a fine thread into the hydrants and securely pinned in place, or carefully locked and caulked in place.

Hose-threads. — Hose-threads on all hydrants to be installed in any given community must of necessity be interchangeable with those already in service, but, where practicable, threads should conform to the National Standard (Figs. 358 to 362).

NATIONAL STANDARD HOSE COUPLING

	INCHES	CENTIMETERS
INSIDE DIAMETER OF HOSE COUPLING	2.5000	6.3500
BLANK END OF MALE PART	0.2500	0.6350
OUTSIDE DIAMETER OF THREAD FINISHED	3.0625	7.7788
DIAMETER OF ROOT OF THREAD	2.8715	7.2936
CLEARANCE BETWEEN MALE AND FEMALE THREADS	0.0300	0.0762
TOTAL LENGTH OF THREADED MALE END	1.0000	2.5400

NUMBER OF THREADS PER INCH........7½
PATTERN OF THREAD........60° V
CUT OFF AT TOP OF THREAD........0.01 OF AN INCH
LEFT IN BOTTOM OF VALLEY........0.01 OF AN INCH
FEMALE END TO BE CUT 0.125 OF AN INCH SHORTER THAN MALE END FOR ENDWISE CLEARANCE.



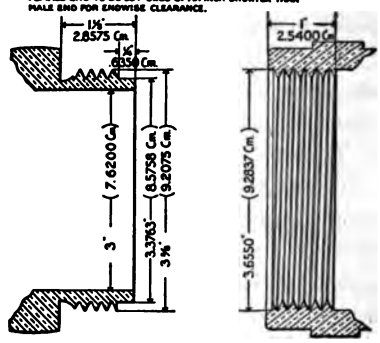
2½" SIZE (6.3500 Cm)

Fig. 358.

NATIONAL STANDARD HOSE COUPLING

	INCHES	CENTIMETERS
INSIDE DIAMETER OF HOSE COUPLING	3.0000	7.6200
BLANK END OF MALE PART	0.2500	0.6350
OUTSIDE DIAMETER OF THREAD FINISHED	3.6250	9.2075
DIAMETER OF ROOT OF THREAD	3.3763	8.5758
CLEARANCE BETWEEN MALE AND FEMALE THREADS	0.0300	0.0762
TOTAL LENGTH OF THREADED MALE END	1.1250	2.8575

NUMBER OF THREADS PER INCH........6
PATTERN OF THREAD........60° V
CUT OFF AT TOP OF THREAD........0.01 OF AN INCH
LEFT IN BOTTOM OF VALLEY........0.01 OF AN INCH
FEMALE END TO BE CUT 0.125 OF AN INCH SHORTER THAN MALE END FOR ENDWISE CLEARANCE.



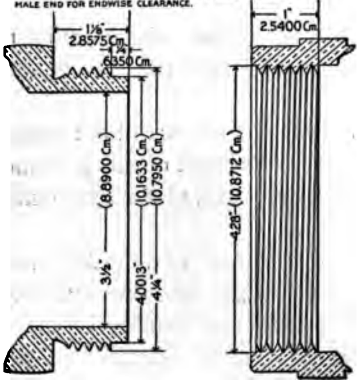
3" SIZE (7.6200 Cm.)

Fig. 359.

NATIONAL STANDARD HOSE COUPLING

	INCHES	CENTIMETERS
INSIDE DIAMETER OF HOSE COUPLING	3.5000	8.8900
BLANK END OF MALE PART	0.2500	0.6350
OUTSIDE DIAMETER OF THREAD FINISHED	4.2500	10.7950
DIAMETER OF ROOT OF THREAD	4.0013	10.1633
CLEARANCE BETWEEN MALE AND FEMALE THREADS	0.0300	0.0762
TOTAL LENGTH OF THREADED MALE END	1.1250	2.8575

NUMBER OF THREADS PER INCH........6
PATTERN OF THREAD........60° V
CUT OFF AT TOP OF THREAD........0.01 OF AN INCH
LEFT IN BOTTOM OF VALLEY........0.01 OF AN INCH
FEMALE END TO BE CUT 0.125 OF AN INCH SHORTER THAN MALE END FOR ENDWISE CLEARANCE.



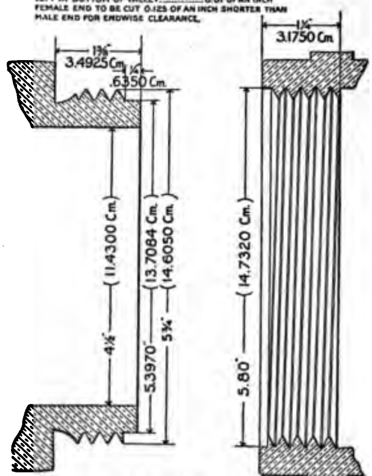
3½" SIZE (8.8900 Cm)

Fig. 360.

NATIONAL STANDARD HOSE COUPLING

	INCHES	CENTIMETERS
INSIDE DIAMETER OF HOSE COUPLING	4.5000	11.4300
BLANK END OF MALE PART	0.2500	0.6350
OUTSIDE DIAMETER OF THREAD FINISHED	5.7500	14.6050
DIAMETER OF ROOT OF THREAD	5.3970	13.7084
CLEARANCE BETWEEN MALE AND FEMALE THREADS	0.0500	0.1270
TOTAL LENGTH OF THREADED MALE END	1.3750	3.4925

NUMBER OF THREADS PER INCH........4
PATTERN OF THREAD........60° V
CUT OFF AT TOP OF THREAD........0.01 OF AN INCH
LEFT IN BOTTOM OF VALLEY........0.01 OF AN INCH
FEMALE END TO BE CUT 0.125 OF AN INCH SHORTER THAN MALE END FOR ENDWISE CLEARANCE.



4½" SIZE (11.4300 Cm.)

Fig. 361.

Hydrant-seat. — The seat must be made of bronze or suitable non-corrodible metal, securely fastened in place.

Gate-valve. — The valve must be faced with a yielding material, such as rubber or leather, except that, if of the gate type, a bronze ring may be used. The valve must be designed so that it can be easily removed for repairs without digging up the hydrant.

Drip-valve. — A positively operating non-corrodible drip-valve must be provided and arranged so as to properly drain the hydrant when the main valve is closed. The seat for the waste-valve, which must be fastened in the hydrant securely, must be made of non-corrodible material. All other parts of the drip mechanism must be so designed as to be easily removed without digging up the hydrant.

Operating-threads. — The operating threads of the hydrant must be so arranged as to do away with the working of any iron or steel parts against iron or steel. Either the operating screw or the operating nut must be made of non-corrodible metal, and sufficiently strong to perform the work for which intended.

Top-nut. — The stem must terminate at the top in a nut of pentagonal shape, finished with slight taper to $1\frac{1}{2}$ in. from point to flat, except for hydrants to be installed where existing hydrants have different shape or size of nut, in which case the additional hydrants must have operating nuts similar to the old one for uniformity. The nut-socket in the wrench must be made without taper, so as to be reversible.

Stuffing-box. — The stuffing-box and gland must be of bronze or suitable non-corrodible metal, or bushed with bronze or suitable non-corrodible metal, when an iron or steel stem is used, or when an iron operating stem nut passes through the stuffing-box. When packing-nut is used, it must be made of bronze or suitable non-corrodible metal. The bottom of the box and end of the gland- or packing-nut must be slightly beveled.

Gland-bolts. — Gland-bolts or stubs must be at least $\frac{1}{2}$ in. in diameter. Bolts or studs may be either of bronze or suitable non-corrodible metal, iron or steel. The nuts must always be of bronze or suitable non-corrodible metal.

Hydrant-top. — The hydrant-top must be designed so as to make the hydrant as weather-proof as possible, and thus overcome the danger from water getting in and freezing around the stem. Provision must be made for oiling both for lubrication and to prevent corrosion. A reasonably tight fit should be made around the stems.

Lettering. — There must be cast on top of the hydrant, in characters raised $\frac{1}{8}$ in., an arrow at least $2\frac{1}{2}$ ins. long, and the word "open" in letters ½ in. high and ⅛ in. in relief, indicating direction to turn to open the hydrant.

Hose-caps. — Hose-caps must be provided for all outlets, and must be securely chained to the barrel with a chain constructed of material not less than ⅛ in. in diameter.

Cap-nut. — The hose-cap nut must be of the same size and shape as the top or operating nut.

Washer in Cap. — When requested by the purchaser, a leather, rubber or lead washer must be provided in the hose-cap, set in a groove to prevent its falling out when the cap is removed.

Markings. — The hydrant must be marked with the name or particular mark of the manufacturer. All letters and figures must be cast on the hydrant barrel above the ground line.

Testing. — Hydrants for pressures of 150 lbs. or less, after being assembled, shall be tested by hydraulic pressure to 300 lbs. per square inch, before leaving the factory. If the working pressure is over 150 lbs. per square inch the hydrants must be tested to twice the working pressure. The test must be made with the valve open, in order to test the whole barrel for porosity, and strength of hydrant body. A second test must be made with valve shut, in order to test the strength and tightness of the valve.

Directions to Open. — Hydrants must open to the left (counter clockwise) except those to be installed where existing hydrants open to the right, in which case the additional hydrants must turn the same as the old ones for the sake of uniformity.

Specifications for Valves

Castings. — All iron castings shall be made from a superior quality of iron, remelted in cupola or air furnace, tough and of even grain, and shall possess a tensile strength of 22,000 lbs. per square inch. The castings must be clean and perfect, without blow or sand holes or defects of any kind. No plugging or stopping of holes will be allowed.

Test Bars. — Specimen bars of the metal used, each being 26 ins. long by 2 ins. wide and 1 in. thick, shall be made without charge as often as the engineer may direct, and in default of definite instructions, the contractor shall make and test at least one bar from each heat or run of metal. The bars when placed flatwise upon supports 24 ins. apart, and loaded in the center, shall support a load of 2200 lbs., and show a deflection of not less than 0.35 of an inch before breaking; or if preferred, tensile bars shall be made which will show a breaking point of not less than 22,000 lbs. per square inch; bars to be cast as nearly as possible to the dimensions without finishing, but corrections may be made by the engineer for variations in width and thickness, and the corrected result must conform to above requirements.

Maker's Name. — Each valve shall have the maker's name cast upon it.

Wrought Iron. — All wrought iron used shall be of the best quality of refined iron, of a tensile strength of at least 45,000 lbs. per square inch.

Composition Metals. — All composition metals to be of the best quality, and, except the stems, to have a tensile strength of not less than 30,000 lbs. per square inch, with 5 per cent elongation in 8 diameters, and 5 per cent reduction of area at breaking point.

Face-joints. — All joints shall be faced true and smooth, so as to make, with suitable gaskets, a perfectly watertight joint.

Fitting and Interchangeable Parts. — The fitting of all parts must be such as make perfect joints, and all parts of the valves of the same make and the same size shall be interchangeable.

Valves to open as specified by the engineer.

Bolts and Nuts. — All bolts and nuts in valves to be made from the best quality of double refined wrought iron or steel, heads, nuts and threads to be standard sizes.

Kind of Valves. — Valves shall be fully mounted with bronze or suitable non-corrodible metal, and be either of the double-disc or made-up gate-type, with bronze or suitable non-corrodible metal mounted wedging devices, or have wedge shaped gates with double faces and seats, designed to work equally well with pressure on either side of the gate. The gates (or discs) shall be of cast iron with bronze or suitable non-corrodible metal faces. These faces shall be machined, dovetailed and driven into corresponding machined grooves in gates (or discs) or riveted on with bronze or suitable non-corrodible metal rivets.

The seats for composition rings in body of valve shall be turned and threaded before rings are screwed in.

Seat- and Gate-rings. — Both seat-rings and gate- (or disc) rings shall have smooth and true faces, and make a perfectly watertight joint.

Valves shall have hub ends suitable for laying with classes B and C American Water Works Association standard pipe. All valves 24 ins. in diameter and larger shall be geared.

By-passes. — Where by-passes are required, they shall, unless otherwise specified, be of the following sizes: 16-in. valve, 3-in. by-pass; 18-in. and 20-in. valves, 3-in. by-pass; 24-in. and 30-in. valves, 4-in. by-pass; 36-in. and 42-in. valves, 6-in. by-pass; 48-in. valves, 8-in. by-pass.

Weight. — Valves without by-passes shall be approximately not less than the following weights for the respective sizes: 3-in., 67 lbs.; 4-in., 85 lbs.; 6-in., 180 lbs.; 8-in., 255 lbs.; 10-in., 400 lbs.; 12-in., 500 lbs.; 14-in., 780 lbs.; 16-in., 900 lbs.; 18-in., 1290 lbs.; 20-in., 1700 lbs.; 24-in., geared, 2750 lbs.; 30-in., geared, 5200 lbs.; 36-in., geared, 8500 lbs.; 42-in., geared, 12,000 lbs.; 48-in., geared, 18,000 lbs.

Valve-stems. — Valve-stems shall be made of solid brass or suitable non-corrodible metal, free from defects, and shall have a tensile strength of not less than 45,000 lbs. per square inch.

Threads. — Threads on stems to be square, acme or $\frac{1}{2}$ V, and cut in most perfect manner, so as to work true and smooth and in perfect line throughout the lift of the valve.

Size of Stems. — Valve stems at the bottom or base of the thread shall not be less than the following sizes in diameter: 3-in. valve, $\frac{44}{64}$ in.; 4-in., $\frac{55}{64}$ in.; 5-in., $\frac{55}{64}$ in.; 6-in., 1 in.; 7-in., 1 in.; 8-in., 1 in.; 9-in., $1\frac{8}{64}$ in.; 10-in., $1\frac{8}{64}$ in.; 12-in., $1\frac{12}{64}$ in.; 14-in., $1\frac{12}{64}$ in.; 16-in., $1\frac{28}{64}$ in.; 18-in., $1\frac{28}{64}$ in.; 20-in., $1\frac{48}{64}$ in.; 22-in., $1\frac{48}{64}$ in.; 24-in., $1\frac{62}{64}$ in.; 30-in., $2\frac{12}{64}$ in.; 36-in., $2\frac{32}{64}$ in.; 42-in., $2\frac{48}{64}$ in.; 48-in., $3\frac{32}{64}$ in.

Wrench-nut. — The wrench nut on stem shall be 2 ins. square with arrow cast on showing direction which valve is to turn to open.

Painting. — All iron work, after being thoroughly cleaned, to be painted throughout with asphaltum varnish, or suitable paint, or dipped in suitable coating material.

Testing. — Valves must be tested for leakage and distortion as follows: On double-disc or made-up gate-type, the body of the valve shall be drilled and tapped with a hole for pipe and a removable plug inserted; through this hole an hydraulic pressure of 300 lbs. per square inch shall be applied; the wedge-shaped gate-type by an hydraulic pressure of 300 lbs. per square inch applied, first between one end and the gate, second between the opposite end and the gate, and third in the bonnet with gate open.

APPENDIX III

MANUFACTURERS' STANDARD SPECIFICATIONS FOR STRUCTURAL AND BOILER STEEL

REVISED *April* 21, 1914

Structural Steel

Grades. — 1. These specifications cover three classes of structural steel, namely:

Class A steel, to be used for railway bridges and ships.
Class B steel, to be used for buildings, highway bridges, train sheds and similar structures.
Class C steel, to be used for structural rivets.

I. MANUFACTURE

Process. — 2. Steel for Classes A and C shall be made by the open-hearth process. Steel for Class **B** may be made either by the open-hearth or by the Bessemer process.

II. CHEMICAL PROPERTIES AND TESTS

Chemical Composition. — 3. The steel shall conform to the following requirements as to chemical composition:

Elements considered	Class A steel	Class B steel	Class C steel
Phosphorus, max., per cent:			
Basic open-hearth	0.04	0.06	0.04
Acid open-hearth	0.06	0.08	0.04
Bessemer	....	0.10	....
Sulphur, max., per cent	0.05	....	0.045

Ladle Analyses. — 4. To determine whether the material conforms to the requirements specified in section 3, an analysis shall be made by the manufacturer from a test ingot taken during the pouring of each melt. A copy of this analysis shall be given to the purchaser or his representative, if requested.

Check Analyses. — 5. A check analysis of Class A and Class C steel may be made by the purchaser from finished material representing each melt, in which case an excess of 25 per cent above the requirements specified in section 3 shall be allowed.

III. PHYSICAL PROPERTIES AND TESTS

Tension Tests. — 6. The steel shall conform to the following requirements as to tensile properties:

Properties considered	Class A steel	Class B steel	Class C steel
Tensile strength, lbs. per sq. in.	55,000–65,000	55,000–65,000*	46,000–56,000
Yield point, minimum, lbs. per sq. in.	0.5 tens. str.	0.5 tens. str.	0.5 tens. str.
Elongation in 8 in., min., per cent	$\frac{1,400,000}{\text{tens. str.}}$†	$\frac{1,400,000}{\text{tens. str.}}$†	$\frac{1,400,000}{\text{tens. str.}}$
Elongation in 2 in., min., per cent (Fig. 363)	22	22	

* See section 8. † See section 9.

Yield Point. — 7. The yield point shall be determined by the drop of the beam of the testing machine.

Modification in Tensile Strength. — 8. Class B steel may have tensile strength up to 70,000 lbs. maximum, provided the elongation is not less than the percentage required for 65,000 lbs. tensile strength.

Modifications in Elongation. — 9. (a) For material over $\frac{3}{4}$ in. in thickness, a deduction of 1 from the percentage of elongation in 8 in. specified for Classes A and B in section 6 shall be made for each increase of $\frac{1}{8}$ in. in thickness above $\frac{3}{4}$ in., to a minimum of 18 per cent.

(b) For material under $\frac{5}{16}$ in. in thickness, a deduction of 2.5 from the percentage of elongation in 8 ins. specified for Classes A and B in section 6 shall be made for each decrease of $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in.

Character of Fracture. — 10. All broken tension test specimens shall show a silky fracture.

Bend Tests. — 11. (a) The test specimen for plates, shapes, and bars shall bend cold through 180 degrees without fracture on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. and under in thickness, flat on itself; for material over $\frac{3}{4}$ in. up to $1\frac{1}{4}$ ins. in thickness, around a pin the diameter of which is equal to $1\frac{1}{2}$ times the thickness of the specimen; and for material over $1\frac{1}{4}$ ins. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(b) The test specimen for pins and rollers shall bend cold through 180 deg. around a 1-in. pin without fracture on the outside of the bent portion.

(c) A rivet rod shall bend cold through 180 degrees flat on itself without fracture on the outside of the bent portion.

(d) Bend tests may be made by pressure or by blows.

Test Specimens. — 12. (a) Tension and bend test specimens shall be taken from the finished rolled or forged product, and shall not be annealed or otherwise treated, except as specified in section 13.

(b) Tension and bend test specimens for plates, shapes and bars, except as specified in paragraph (c), shall be of the full thickness of

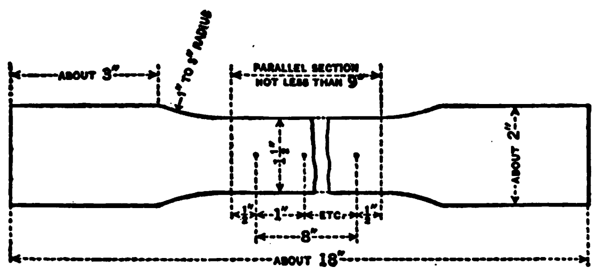


FIG. 362.

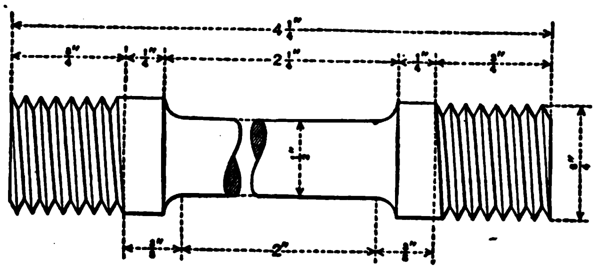


FIG. 363.

material as rolled, and with both edges milled to the form and dimensions shown in Fig. 362, or may have both edges parallel.

(c) Tension and bend test specimens for plates and bars (except eye-bar flats) over $1\frac{1}{2}$ ins. in thickness or diameter may be turned or planed to a diameter or thickness of at least $\frac{3}{4}$ in. for a length of at least 9 in.

(d) Tension and bend test specimens for pins and rollers shall be taken parallel to the axis, 1 in. from the surface of the bar. Tension test specimens shall be of the form and dimensions shown in Fig. 363. Bend test specimens shall be 1 in. by $\frac{1}{2}$ in. in section.

(e) Rivet bars shall be tested in full-size section as rolled.

Annealed Specimens. — 13. Test specimens for material which is to be annealed or otherwise treated before use, shall be cut from properly annealed or similarly treated short lengths of the full section of the piece.

Number of Tests. — 14. (a) At least one tension test and one bend test shall be made from each melt. If material from one melt differs $\frac{3}{8}$ in. or more in thickness, tests shall be made from both the thickest and the thinnest material rolled.

(b) If any test specimen develops flaws, or if an 8-in. tension test specimen breaks outside the middle third of the gauge length, or if a 2-in. tension test specimen breaks outside the gauge length, it may be discarded and another specimen substituted therefor.

(c) Material intended for fillers or ornamental purposes will not be subject to test.

IV. PERMISSIBLE VARIATIONS IN WEIGHT AND GAUGE

Permissible Variations. — 15. (a) The sectional area or weight of each structural shape and of each rolled-edge plate up to and including 36 ins. in width, shall not vary more than 2.5 per cent from theoretical or specified amounts.

(b) The thickness or weight of each universal plate over 36 ins. in width, and of each sheared plate, shall conform to the schedules of permissible variations for sheared plates, Manufacturers' Standard Practice, appended to these specifications.

(c) The weights of angles, tees, zees, and channels of bar sizes, and the dimensions of rounds, squares, hexagons, and flats, shall conform to the Manufacturers' Standard Practice governing the allowable variations in size and weight of hot-rolled bars.

V. FINISH

Finish. — 16. The finished material shall be free from injurious defects, and shall have a workmanlike finish.

VI. MARKING

Marking. — 17. The name of the manufacturer and the melt number shall be legibly marked, stamped, or rolled upon all finished material, except that each pin and roller shall be stamped on the end. Rivet and lattice steel and other small pieces may be shipped in securely fastened bundles, with the above marks legibly stamped on attached metal tags. Test specimens shall have their melt numbers plainly marked or stamped.

VII. INSPECTION AND REJECTION

Inspection. — 18. The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests and inspection shall be made at the place of manufacture prior to shipment, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

Rejection. — 19. Material which, subsequent to the above tests at the mills and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, may be rejected at the shop, and shall then be replaced by the manufacturer at his own cost.

Boiler Steel

Grades. — 1. There shall be three grades of steel for boilers, namely: flange, firebox, and boiler rivet.

I. MANUFACTURE

Process. — 2. The steel shall be made by the open-hearth process.

II. CHEMICAL PROPERTIES AND TESTS

Chemical Composition. — 3. The steel shall conform to the following requirements as to chemical composition:

Elements considered	Flange steel	Firebox steel	Boiler rivet steel
Manganese, per cent	0.30–0.60	0.30–0.50	0.30–0.50
Phosphorus, max., per cent:			
Basic	0.04	0.035	0.04
Acid	0.05	0.04	0.04
Sulphur, max., per cent	0.05	0.04	0.045

Ladle Analyses. — 4. To determine whether the material conforms to the requirements specified in section 3, an analysis shall be made by the manufacturer from a test ingot taken during the pouring of each melt. A copy of this analysis shall be given to the purchaser or his representative.

Check Analyses. — 5. A check analysis may be made by the purchaser from a broken tension test specimen representing each plate as rolled, and this analysis shall conform to the requirements specified in section 3.

III. PHYSICAL PROPERTIES AND TESTS

Tension Tests. — 6. The steel shall conform to the following requirements as to tensile properties:

Properties considered	Flange steel	Firebox steel	Boiler rivet steel
Tensile strength, lbs. per sq. in..	55,000–65,000	52,000–60,000	45,000–55,000
Yield point, min., lbs. per sq. in.	0.5 tens. str.	0.5 tens. str.	0.5 tens. str.
Elongation in 8 in., min., per cent...........................	$\frac{1,450,000}{\text{tens. str.}}$*	$\frac{1,450,000}{\text{tens. str.}}$*	$\frac{1,450,000}{\text{tens. str.}}$

* See section 8.

Yield Point. — 7. The yield point shall be determined by the drop of the beam of the testing machine.

Modifications in Elongation. — 8. (a) For plates over $\frac{3}{4}$ in. in thickness, a deduction of 0.5 from the specified percentage of elongation will be allowed for each increase of $\frac{1}{8}$ in. in thickness above $\frac{3}{4}$ in., to a minimum of 20 per cent.

(b) For plates under $\frac{5}{16}$ in. in thickness, a deduction of 2.5 from the percentage of elongation specified in section 6 shall be made for each decrease of $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in.

Bend Tests. — 9. (a) Cold-bend tests shall be made on the material as rolled.

(b) Quench-bend test specimens, before bending, shall be heated to a light cherry red as seen in the dark (about 1200 degrees F.), and quenched in water the temperature of which is about 80 degrees F.

(c) Specimens for cold-bend and quench-bend tests of flange and firebox steel shall bend through 180 degrees without fracture on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. and under in thickness, flat on themselves; for material over $\frac{3}{4}$ in. up to $1\frac{1}{4}$ ins. in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material over $1\frac{1}{4}$ ins. in thickness, around a pin the diameter of which is equal to $1\frac{1}{2}$ times the thickness of the specimen.

(d) Specimens for cold-bend and quench-bend tests of boiler rivet steel shall bend cold through 180 degrees flat on themselves without fracture on the outside of the bent portion.

(e) Bend tests may be made by pressure or by blows.

Test Specimens. — 10. (a) Tension and bend test specimens for plates shall be taken from the finished product, and shall be of the full thickness of material as rolled. Tension test specimens shall be of the form and dimensions shown in Fig. 362. Bend test specimens shall be $1\frac{1}{2}$ ins. to $2\frac{1}{2}$ ins. wide, and shall have the sheared edges milled or planed.

(b) The tension and bend test specimens for rivet bars shall be of the full-size section of material as rolled.

Number of Tests. — 11. (a) One tension, one cold-bend, and one quench-bend test shall be made from each plate as rolled.

(*b*) Two tension, two cold-bend, and two quench-bend tests shall be made for each melt of rivet steel.

(*c*) If any test specimen develops flaws, or if a tension test specimen breaks outside the middle third of the gauge length, it may be discarded and another specimen substituted therefor.

IV. PERMISSIBLE VARIATIONS IN WEIGHT AND GAUGE

Permissible Variations. — 12. (a) The thickness or weight of each sheared plate shall conform to the schedule of permissible variations, Manufacturers' Standard Practice, appended to these specifications.

(b) The dimensions of rivet bars shall conform to the Manufacturers' Standard Practice governing allowable variations in the size of hot-rolled bars.

V. FINISH

Finish. — 13. The finished material shall be free from injurious defects, and shall have a workmanlike finish.

VI. MARKING

Marking. — 14. The melt or slab number, name of the manufacturer, grade, and the minimum tensile strength for its grade as specified in section 6 shall be legibly stamped on each plate. The melt or slab number shall be legibly stamped on each test specimen representing that melt or slab.

VII. INSPECTION AND REJECTION

Inspection. — 15. The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests and inspection shall be made at the place of manufacture prior to shipment, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

Rejection. — 16. Material which, subsequent to the above tests at the mills and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, may be rejected at the shop, and shall then be replaced by the manufacturer at his own cost.

MANUFACTURERS' STANDARD PRACTICE

Permissible Variations in Weight and Thickness of Sheared Plates

One cubic inch of rolled steel is assumed to weigh 0.2833 lb.

When ordered to weight per square foot: The weight of each lot* in each shipment shall not vary from the weight ordered more than the amount given in the following table:

Ordered weight, lbs. per sq. ft.	Permissible variations in average weights per square foot of plates for widths given, expressed in percentages of ordered weights																		Ordered weight, lbs. per sq. ft.
	Under 48 ins.		48 to 60 ins. exclusive		60 to 72 ins. exclusive		72 to 84 ins. exclusive		84 to 96 ins. exclusive		96 to 108 ins. exclusive		108 to 120 ins. exclusive		120 to 132 ins. exclusive		132 ins. or over		
	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	
Under 5	5	3	5.5	3	6	3	7	3	..	...	..	...	..	...	..	...	..	...	Under 5
5 to 7.5 excl.	4.5	3	5	3	5.5	3	6	3	..	...	..	...	..	...	..	...	..	...	5 to 7.5 excl.
7.5 " 10 "	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	..	...	..	...	7.5 " 10 "
10 " 12.5 "	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	9	3	10 " 12.5 "
12.5 " 15 "	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	12.5 " 15 "
15 " 17.5 "	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	15 " 17.5 "
17.5 " 20 "	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	17.5 " 20 "
20 " 25 "	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	20 " 25 "
25 " 30 "	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	5	3	25 " 30 "
30 " 40 "	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	30 " 40 "
40 or over	2	2	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	40 or over

* The term "lot" applied to this table means all of the plates of each group width and group weight.

NOTE. — The weight per square foot of individual plates shall not vary from the ordered weight by more than $1\frac{1}{3}$ times the amount given in this table.

MANUFACTURERS' STANDARD PRACTICE

Permissible Variations in Weight and Thickness of Sheared Plates

One cubic inch of rolled steel is assumed to weigh 0.2833 lb.

When ordered to thickness: The thickness of each plate shall not vary more than 0.01 in. under that ordered. The overweight of each lot* in each shipment shall not exceed the amount given in the following table:

Ordered thickness, inches	Permissible excess in average weights per square foot of plates for widths given, expressed in percentages of nominal weights									Ordered thickness, inches
	Under 48 ins.	48 to 60 ins. exclusive	60 to 72 ins. exclusive	72 to 84 ins. exclusive	84 to 96 ins. exclusive	96 to 108 ins. exclusive	108 to 120 ins. exclusive	120 to 132 ins. exclusive	132 ins. or over	
Under $\frac{1}{8}$	9	10	12	14	....	....	....	....	....	Under $\frac{1}{8}$
$\frac{1}{8}$ to $\frac{3}{16}$ excl.	8	9	10	12	....	....	....	....	....	$\frac{1}{8}$ to $\frac{3}{16}$ excl.
$\frac{3}{16}$ " $\frac{1}{4}$ "	7	8	9	10	12	....	....	....	....	$\frac{3}{16}$ " $\frac{1}{4}$ "
$\frac{1}{4}$ " $\frac{5}{16}$ "	6	7	8	9	10	12	14	16	19	$\frac{1}{4}$ " $\frac{5}{16}$ "
$\frac{5}{16}$ " $\frac{3}{8}$ "	5.	6	7	8	9	10	12	14	17	$\frac{5}{16}$ " $\frac{3}{8}$ "
$\frac{3}{8}$ " $\frac{7}{16}$ "	4.5	5	6	7	8	9	10	12	15	$\frac{3}{8}$ " $\frac{7}{16}$ "
$\frac{7}{16}$ " $\frac{1}{2}$ "	4	4.5	5	6	7	8	9	10	13	$\frac{7}{16}$ " $\frac{1}{2}$ "
$\frac{1}{2}$ " $\frac{5}{8}$ "	3.5	4	4.5	5	6	7	8	9	11	$\frac{1}{2}$ " $\frac{5}{8}$ "
$\frac{5}{8}$ " $\frac{3}{4}$ "	3	3.5	4	4.5	5	6	7	8	9	$\frac{5}{8}$ " $\frac{3}{4}$ "
$\frac{3}{4}$ " 1 "	2.5	3	3.5	4	4.5	5	6	7	8	$\frac{3}{4}$ " 1 "
1 or over	2.5	2.5	3	3.5	4	4.5	5	6	7	1 or over

* The term "lot" applied to this table means all of the plates of each group width and group thickness.

APPENDIX IV

SPECIAL FIRE PROTECTION FOR BUILDINGS*

In large buildings, used for offices, factories, etc., special provision should be made for extinguishing fires as soon as they start. Stand-pipes to which fire-hose with nozzles are attached at each floor are usually placed in such buildings, and are connected with the public water supply, or with a water tank, etc. During the past 30 years various systems of automatic sprinklers have been devised. The principal feature of these inventions is a system of pipes with automatic sprinklers, placed below the ceilings of the different floors. The pipes are usually kept full of water under pressure. By the heat of the fire causing the melting of fusible solder, used in their construction, the sprinklers open automatically, and discharge water near the site of the fire. Short descriptions of these systems are given below. While the details are usually arranged according to the rules of the *National Board of Fire Underwriters*, the connections with the public pipe system come under the supervision of the engineer or superintendent in charge of the public water-works.

Stand-pipe and Hose.—For a building of ordinary height, the stand-pipes are usually 2 to 4 ins. in diameter, and are connected on each floor to $1\frac{1}{2}$-in. hose with $\frac{1}{2}$- to $\frac{3}{4}$-in. nozzles of galvanized iron or brass.

For higher buildings, and where Siamese connections for fire-engines are provided at the street level, the stand-pipes should be 6 to 8 ins. in diameter, and the hose attached at each floor should be $2\frac{1}{2}$ ins. in diameter, with play pipe having 1-in. or $1\frac{1}{8}$-in. tip. The hose should be of the best quality of linen, in 50-ft. lengths, and should be kept in flat folds on pin racks, and not on reels, which would make the inspection difficult. In city buildings having fire-proof stair-shafts, the stand-pipes are usually placed in these shafts.

The stand-pipes should always be full of water with at least 30 lbs. pressure per square inch at the highest level. As already stated, the supply is obtained from the public mains, or from a storage tank. In some buildings special pumps are provided, to maintain the required pressure in the stand-pipes until the fire-engines arrive.

The perforated pipe system, originally devised in England, and in troduced into the United States about 1852, was the first step in-

* For full descriptions of the means used for protecting buildings against fires — such as automatic sprinklers, automatic fire alarms, etc. — see *Automatic Sprinkler Protection* by Gorham Dana, S.B., Boston, Mass., 1914; Crosby-Fiske Handbook of Fire Protection, Louisville, Ky., 1914; also, *Fire Prevention and Protection* by A. C. Hutson, C. E., 3d Edition, New York, 1916.

automatic fire protection. Various improvements were made in this system by James B. Francis, M. Am. Soc. C. E., who used it in 1859 in mills in Lowell, Mass. Francis placed the pipes close to the ceiling, running across the mill in the center of each bay. The pipes were perforated with $\frac{1}{10}$-in. holes which were placed 9 ins. apart, alternately on different sides of the pipe and at a point a little above its horizontal center. By this arrangement the water wet the ceilings, as well as the floors, as soon as the gate-valve at the stand-pipe was opened. In order to reduce the friction loss of the water in the pipes, it was found necessary to make the area of the pipe at any point about twice the area of the perforations which were to be supplied. Various improvements in the perforated pipe system were made in 1870–1880 by W. B. Whiting, Frederick Grinnell, Hall Brothers, etc.

Pipe-scheme. — In all of the early perforated pipe systems, the same general scheme of piping was used. Beginning at the end of a room, parallel lines of pipe, 10 ft. or less apart, were placed close to the ceiling, the diameter of the pipes being $\frac{3}{4}$ in. at the end of each line and increasing according to the rule given above. These lines of pipe connected with a feed-pipe, which in turn was enlarged until the riser, or upright pipe feeding the entire floor, was reached. The riser passed down through the floors or on the outside of the building to a controlling valve. A separate riser with controlling valve was provided for each floor and the valves were located in a group, being labeled. In the best systems these valves were placed in a valve-house at a safe distance from the building.

Some of the defects of the perforated pipe system are that the small perforations in the pipes are apt to be closed by rust, paint, or sediment, that the water is not concentrated at the points where it is most needed, and that the system cannot be tested, for evident reasons. Trouble occurred occasionally with this system by the valves being opened through error or malice, or by their leaking. In spite of these defects, the perforated pipe system was extensively used from 1852 to about 1885, especially in New England cotton mills, until it was superseded by automatic sprinklers.

Automatic sprinkler systems consist of an arrangement of pipes to which are attached, at regular intervals of about 10 ft., sprinklers, called also sprinkler-heads, which are released at a fixed temperature of about 160 degrees Fahrenheit, and distribute a stream of water. In all modern sprinklers this releasing is accomplished by the melting of low-fusing solder, which is used in the construction of the device. In some of the older types of sprinkler heads, the releasing was done by the burning of a cord, the explosion of gunpowder, the expansion of a volatile liquid in a closed receptacle, or the expansion of wax. None of these devices proved to have practical value and, at present, fusible solder alone is used for releasing the sprinklers.

Automatic Sprinklers. — The first automatic sprinkler-head was invented in 1864 by Major A. Stewart Harrison * of the British Army. This invention, which possessed considerable merit, was put on the market without being patented. Modifications and improvements in this device were soon made, especially in the United States. Since 1872 more than 450 United States patents for automatic sprinklers have been taken out. Some of these inventions were good sprinkler-heads, but were soon superseded by better devices. Out of all of these automatic sprinklers, only about twelve are approved, at the present time, by the *National Board of Fire Underwriters.*

Frederick Grinnell, of Providence, R. I., invented in 1882 a sprinkler which was a radical departure from the types existing at that time. Instead of using a nozzle for the sprinkler jet, he adopted an orifice, $\frac{5}{16}$ in. in diameter, formed in a thin brass plate or diaphragm, which was inserted in an enlargement of the sprinkler casting (Fig. 364). The head is pendent, and in order to strengthen the thin brass diaphragm, a stiff spring-plate is inserted, just under it, so as to insure a strong spring action to the levers, and to prevent the diaphragm from collapsing when the sprinkler is not under pressure. The edges of the orifice in the diaphragm are bent over to form a seat ring about $\frac{1}{8}$ in. wide. The orifice is covered by a valve, which is made in one piece with a deflector having projecting teeth, at right angles, around the edge, for distributing the water. The center of the deflector is depressed and filled with lead to form the valve. A pair of compound levers serve to hold the valve in place. The first lever is held at one end by a notch in the yoke, and at the other end by the second lever. The latter is hooked under a notch in the other side of the yoke, and soldered at the extreme lower end by fusible solder, reinforced by an L-shaped piece of wire. The melting of the solder releases the levers and the valve, thus putting the sprinkler into operation.

FIG. 364. — Grinnell Sprinkler.

The Grinnell head soon became a standard sprinkler and was extensively used. It is one of the few early sprinklers which, after numerous improvements, is still on the market.

* "Automatic Sprinkler Protection," by Gorham Dana, S.B., p. 13, Boston, 1914.

In Fig. 365 some other standard sprinkler heads, in which the valve closing the sprinkler opening is held in place by levers which are locked by links of fusible solder, are shown.* The usual formula for this solder is as follows:

Bismuth, 4 parts; lead, 2 parts; cadmium, 1 part; tin, 1 part.

The solder melts at a temperature of 160–165 degrees Fahrenheit.

Pipe Schemes. — Several different arrangements of distributing pipes have been used with automatic sprinkler heads. Henry S. Parmelee devised the so-called *tree system of pipes*. He placed his

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FIG. 365. — Auto-sprinklers.

main feed pipes about 20 ft. apart, and connected to them, at intervals of about 10 ft., $\frac{3}{4}$-in. branch pipes, each 5 ft. long. This brought the sprinkler heads about 10 ft. apart in each direction. By this arrangement each sprinkler was placed on a *dead end* and was not cooled by water flowing past it during a fire. The feed pipe was enlarged, where the branch lines connected, so as to provide sufficient capacity to feed practically all of the sprinklers on a floor, at the same time. Only one stand-pipe was used for all of the floors, and it was made large enough to supply all of the sprinklers on one floor simultaneously. This was based on the theory, still accepted, that, with rare exceptions, only one floor will be on fire at a time.

Fig. 366 shows other approved plans for risers, feed pipes, and sprinklers. An arrangement of open sprinklers with one line on the outside of the building under the cornice is sometimes used.

Two methods of operating sprinkler system, known respectively as the *wet* and the *dry*, are in use. In the former, which is the safer arrangement, the whole sprinkler system is constantly full of water

* For descriptions of other standard sprinkler heads see "Automatic Sprinkler Protection," by Gorham Dana, S.B., Boston, 1914.

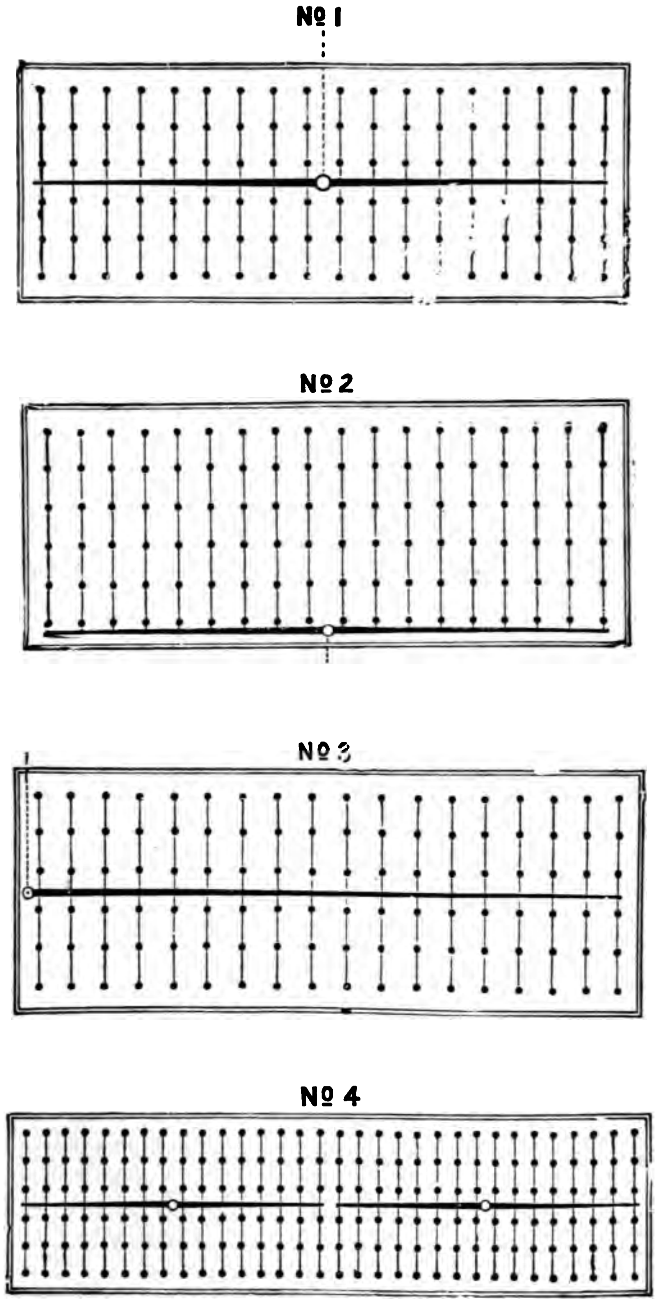

FIG. 366. — Pipe Systems for Auto-sprinklers.

under pressure. In the latter system the water is only turned on in case of fire.

The spacing of the sprinklers varies greatly according to the construction of the ceiling under which they are placed. As a general rule, the sprinklers should be so located that each will not have to cover over 80–100 sq. ft. of floor area. The sprinklers should point upwards, except in special cases. The objections to having them point downwards are: They are apt to get injured by blows; they become clogged with sediment; are less sensitive to heat as the solder is farther from the ceiling, and they cannot be drained.

Pipes. — The sizes of pipes used to feed the sprinklers followed, at first, closely the sizes used for the perforated pipe systems. The standard schedule of pipe adopted by the *National Fire Protection Association* in 1905, and now used in all parts of the country, is given in the following table:

PIPE SCHEDULE

Diameter of pipe, inches	Number of heads	Diameter of pipe, inches	Number of heads
$\frac{3}{4}$	1	3	36
1	2	$3\frac{1}{2}$	55
$1\frac{1}{4}$	3	4	80
$1\frac{1}{2}$	5	5	140
2	10	6	200
$2\frac{1}{2}$	20	....	....

The above table is based upon the supposition that only one floor will be on the fire at a time. A fire that will open a large number of heads or more than one floor, at a time, cannot usually be controlled by sprinklers.

Valves and Fittings. — A suitable gate-valve must be provided for every sprinkler system, for shutting off the water after a fire, or in case of leakage or repairs. The best type of valve for this purpose is the *outside screw and yoke gate* which shows clearly at a distance, by the position of the stem, whether the valve is open or closed, The shut-off valve should be placed as far as possible from the building in which the sprinkler system is installed, so as to be accessible, even in the case of a great conflagration. In the great fire at Salem, some of these valves could not be reached, owing to their proximity to the burning buildings, and the water could not, therefore, be shut off, in some cases, after the fire had stopped. Nor could the street main be closed in the fire referred to, as the service-pipes had been connected directly to a 20-in. main which was one of the main supply pipes of the city. This shows the danger of tapping the *main feeders* of a city's pipe system. In such cases a smaller parallel main, which could easily be shut off in case of necessity, should be laid for house connections.

When a sprinkler system is fed from two different sources, a check-valve must be placed in each, to prevent the water from flowing from one source to the other. The best kind of valve for this purpose is the *straightway swing valve,* in which a disc is hung from a pivot in such a manner that it rests normally against the valve seat, which is placed at a slight angle from the vertical, and only opens when the water comes from one direction.

Steamer connections should be provided where fire-engines are available, especially where the other supplies have not much pressure. Their use is generally restricted to large cities. These connections are usually made for 2½- or 3-inch hose, and each should be provided with a stop-gate and a check-valve. Frequently these connections are made double (Siamesed) (Fig. 367), in which case each inlet must be check-valved against the other.

FIG. 367. — Steamer Connection.

Automatic fire alarm valves should be connected with all automatic sprinkler systems. Harrison, who invented the first sprinkler-head, planned to have an alarm valve in the sprinkler system, which was to be actuated by the flow of water, when a sprinkler was in operation. Parmelee also devised an automatic alarm valve, which consisted of a flapper check-valve, placed in the main riser near its base. A lever, connected with the hinged end of the check-valve, extended through a stuffing-box, and was connected by a wire to a steam whistle, or to a mechanical gong, which was put into operation the moment the check-valve was lifted by the flow of water.

Many improvements have been made in these devices. One of them consists in making the alarm valve close an electric circuit, which causes an electric bell to sound in a fire-engine house, or some other building.

APPENDIX V

FIRE STREAM TABLES

NATIONAL BOARD OF FIRE UNDERWRITERS

EFFECTIVE REACH OF FIRE STREAMS

Showing the Distance in Feet from the Nozzle at which Streams will do Effective Work with a Moderate Wind Blowing. With a Strong Wind the Reach is Greatly Reduced.

Pressure at nozzle	Size of nozzle									
	1-inch		1⅛-inch		1¼-inch		1⅜-inch		1½-inch	
	Vertical distance, feet	Horizontal distance, feet	Vertical distance, feet	Horizontal distance, feet	Vertical distance, feet	Horizontal distance, feet	Vertical distance, feet	Horizontal distance, feet	Vertical distance, feet	Horizontal distance, feet
20	35	37	36	38	36	39	36	40	37	42
25	43	42	44	44	45	46	45	47	46	49
30	51	47	52	50	52	52	53	54	54	56
35	58	51	59	54	59	58	60	59	62	62
40	64	55	65	59	65	62	66	64	69	66
45	69	58	70	63	70	66	72	68	74	71
50	73	61	75	66	75	69	77	72	79	75
55	76	64	79	69	80	72	81	75	83	78
60	79	67	83	72	84	75	85	77	87	80
65	82	70	86	75	87	78	88	79	90	82
70	85	72	88	77	90	80	91	82	92	84
75	87	74	90	79	92	82	93	84	94	86
80	89	76	92	81	94	84	95	86	96	88
85	91	78	94	83	96	87	97	88	98	90
90	92	80	96	85	98	89	99	90	100	91

NOTE.—Nozzle pressures are as indicated by Pitot tube. The horizontal and vertical distances are based on experiments by Mr. John R. Freeman, *Transactions*, Am. Soc. C. E., Vol. XXI.

FRICTION LOSS IN FIRE HOSE

Based on Tests of Best Quality Rubber-lined Fire Hose *

Flow, gallons per minute	Pressure loss in each 100 feet of hose, pounds per square inch				Flow, gallons per minute	Pressure loss in each 100 feet of hose, pounds per square inch		
	2½-in. hose	3-in. hose	3½-in. hose	2 lines of 2½-in. Siamesed		3-in. hose	3½-in. hose	2 lines of 2½-in. Siamesed
140	5.2	2.0	0.9	1.4	525	23.2	10.5	16.6
160	6.6	2.6	1.2	1.9	550	25.2	11.4	18.1
180	8.3	3.2	1.5	2.3	575	27.5	12.4	19.0
200	10.1	3.9	1.8	2.8	600	29.9	13.4	21.2
220	12.0	4.2	2.1	3.3	625	32.0	14.4	23.0
240	14.1	5.4	2.5	3.9	650	34.5	15.5	24.8
260	16.4	6.3	2.9	4.5	675	37.0	16.6	26.5
280	18.7	7.2	3.3	5.2	700	39.5	17.7	28.3
300	21.2	8.2	3.7	5.9	725	42.3	18.9	30.2
320	23.8	9.3	4.2	6.6	750	45.0	20.1	32.2
340	26.9	10.5	4.7	7.4	775	47.8	21.4	34.2
360	30.0	11.5	5.2	8.3	800	50.5	22.7	36.2
380	33.0	12.8	5.8	9.2	825	53.5	24.0	38.4
400	36.2	14.1	6.3	10.1	850	56.5	25.4	40.7
425	40.8	15.7	7.0	11.3	875	59.7	26.8	43.1
450	45.2	17.5	7.9	12.5	900	63.0	28.2	45.2
475	50.0	19.3	8.7	13.8	1000	76.5	34.3	55.0
500	55.0	21.2	9.5	15.2	1100	91.5	41.0	65.5

*** Rough rubber lining is liable to increase the losses given in the table as much as 50 per cent.**

DISCHARGE TABLE FOR SMOOTH NOZZLES

Nozzle Pressure Measured by Pitot Gage

Nozzle pressure in lbs. per sq. inch	Nozzle diam. in inches 1	1⅛	1¼	1⅜	1½	Nozzle pressure in lbs. per sq. inch	Nozzle diam. in inches 1	1⅛	1¼	1⅜	1½
	Gallons per minute						Gallons per minute				
5	66	84	103	125	149	60	229	290	357	434	517
6	72	92	113	137	163	62	233	295	363	441	525
7	78	99	122	148	176	64	237	299	369	448	533
8	84	106	131	158	188	66	240	304	375	455	542
9	89	112	139	168	200	68	244	308	381	462	550
10	93	118	146	177	211	70	247	313	386	469	558
12	102	130	160	194	231	72	251	318	391	475	566
14	110	140	173	210	249	74	254	322	397	482	574
16	118	150	185	224	267	76	258	326	402	488	582
18	125	159	196	237	283	78	261	330	407	494	589
20	132	167	206	250	298	80	264	335	413	500	596
22	139	175	216	263	313	82	268	339	418	507	604
24	145	183	226	275	327	84	271	343	423	513	611
26	151	191	235	286	340	86	274	347	428	519	618
28	157	198	244	297	353	88	277	351	433	525	626
30	162	205	253	307	365	90	280	355	438	531	633
32	167	212	261	317	377	92	283	359	443	537	640
34	172	218	269	327	389	94	286	363	447	543	647
36	177	224	277	336	400	96	289	367	452	549	654
38	182	231	285	345	411	98	292	370	456	554	660
40	187	237	292	354	422	100	295	374	461	560	667
42	192	243	299	363	432	105	303	383	473	574	683
44	196	248	306	372	442	110	310	392	484	588	699
46	200	254	313	380	452	115	317	401	495	600	715
48	205	259	320	388	462	120	324	410	505	613	730
50	209	265	326	396	472	125	331	418	516	626	745
52	213	270	333	404	481	130	337	427	526	638	760
54	217	275	339	412	490	135	343	435	536	650	775
56	221	280	345	419	499	140	350	443	546	662	789
58	225	285	351	426	508	145	356	450	556	674	803
60	229	290	357	434	517	150	362	458	565	686	817

Nozzle pressure in lbs. per sq. inch	Nozzle diam. in inches 1⅝	1¾	1⅞	2	2¼	Nozzle pressure in lbs. per sq. inch	Nozzle diam. in inches 1⅝	1¾	1⅞	2	2¼
	Gallons per minute						Gallons per minute				
5	175	203	234	266	337	60	607	704	810	920	1167
6	192	223	256	292	369	62	617	716	823	936	1187
7	207	241	277	315	399	64	627	727	836	951	1206
8	222	257	296	336	427	66	636	738	850	965	1224
9	235	273	314	357	452	68	646	750	862	980	1242
10	248	288	330	376	477	70	655	761	875	994	1260
12	271	315	362	412	522	72	665	771	887	1008	1278
14	293	340	391	445	564	74	674	782	900	1023	1296
16	313	364	418	475	603	76	683	792	911	1036	1313
18	332	386	444	504	640	78	692	803	924	1050	1330
20	350	407	468	532	674	80	700	813	935	1063	1347
22	367	427	490	557	707	82	709	823	946	1076	1364
24	384	446	512	582	739	84	718	833	959	1089	1380
26	400	464	533	606	769	86	726	843	970	1102	1396
28	415	481	554	629	799	88	735	853	981	1115	1412
30	429	498	572	651	826	90	743	862	992	1128	1429
32	443	514	591	673	854	92	751	872	1002	1140	1445
34	457	530	610	693	880	94	759	881	1012	1152	1460
36	470	546	627	713	905	96	767	890	1022	1164	1476
38	483	561	645	733	930	98	775	900	1032	1176	1491
40	496	575	661	752	954	100	783	909	1043	1189	1506
42	508	589	678	770	978	105	803	932	1070	1218	1542
44	520	603	694	788	1000	110	822	954	1095	1247	1579
46	531	617	710	806	1021	115	840	975	1120	1275	1615
48	543	630	725	824	1043	120	858	996	1144	1303	1649
50	554	643	740	841	1065	125	876	1016	1168	1329	1683
52	565	656	754	857	1087	130	893	1036	1191	1356	1717
54	576	668	769	873	1108	135	910	1056	1213	1382	1750
56	586	680	782	889	1129	140	927	1076	1235	1407	1780
58	596	692	796	905	1149	145	944	1095	1257	1432	1812
60	607	704	810	920	1168	150	960	1114	1279	1456	1843

Assumed coefficient of discharge per cent = 0.995 0.995 0.996 0.997 0.997   Assumed coefficient of discharge per cent = 0.99 0.99 0.99 0.99½ 0.99½

NOTE. — Coefficients of discharge are based on experiments by Mr. John R. Freeman, *Transactions* Am. Soc. C. E., Vols. XXI and XXIV.

1-INCH SMOOTH NOZZLE. — 2½- AND 3-INCH HOSE

Pressures required at hydrant or fire engine, while stream is flowing to maintain nozzle pressures given in first column, through various lengths of best quality 2½- and 3-inch rubber-lined hose

Nozzle pressure indicated by Pitot gauge	Discharge, gallons per minute	Single 2½-inch lines										Single 3-inch lines				Two 2½-inch lines Siamesed			Nozzle pressure indicated by Pitot gauge
		100 feet	200 feet	300 feet	400 feet	500 feet	600 feet	700 feet	800 feet	1000 feet	1200 feet	800 feet	1000 feet	1200 feet	1500 feet	1000 feet	1500 feet	2000 feet	
20	132	25	30	35	39	44	49	53	58	68	77	35	39	42	48	33	40	46	**20**
25	148	31	37	43	49	55	60	66	72	84	95	43	48	52	59	41	49	57	**25**
30	162	38	44	51	58	65	72	78	85	99	112	52	57	62	70	49	59	68	**30**
35	175	44	52	59	67	75	83	91	98	114	130	60	66	72	81	57	68	79	**35**
40	187	50	59	68	77	86	94	103	112	130	148	68	75	82	92	65	78	90	**40**
45	198	56	66	76	86	96	106	115	125	145	165	77	84	92	103	72	86	99	**45**
50	209	62	73	84	95	106	117	128	139	160	182	85	93	102	114	80	95	110	**50**
55	219	68	80	92	104	116	128	140	152	175	199	93	102	112	125	88	105	121	**55**
60	229	75	88	101	114	127	140	153	166	192	218	102	112	122	137	96	114	132	**60**
65	238	81	95	109	123	137	151	165	179	207	235	110	121	131	148	103	122	141	**65**
70	247	87	102	117	132	147	162	177	192	222	252	118	130	141	159	111	132	152	**70**
75	256	93	109	125	141	157	173	189	205	237	269	127	139	151	170	120	142	164	**75**
80	264	99	116	133	150	167	183	200	217	251	285	135	148	161	181	128	151	175	**80**
85	272	105	123	141	159	177	195	212	230	266	302	143	156	170	191	135	159	184	**85**
90	280	111	130	149	167	186	205	224	243	280	...	151	165	180	202	143	169	195	**90**
95	287	117	137	157	177	196	216	236	256	295	...	158	173	189	211	150	177	204	**95**
100	295	123	144	165	185	206	227	247	268	310	...	167	183	199	223	157	186	215	**100**

1⅛-INCH SMOOTH NOZZLE. — 2½- AND 3-INCH HOSE

Nozzle pressure indicated by Pitot gauge	Discharge, gallons per minute	Pressures required at hydrant or fire engine, while stream is flowing, to maintain nozzle pressures given in first column, through various lengths of best quality 2½- and 3-inch rubber-lined hose																						Nozzle pressure indicated by Pitot gauge
		Single 2½-inch lines										Single 3-inch lines							Two 2½-inch lines Siamesed					
		100 feet	200 feet	300 feet	400 feet	500 feet	600 feet	700 feet	800 feet	1000 feet	1200 feet	400 feet	600 feet	800 feet	1000 feet	1200 feet	1500 feet	1800 feet	800 feet	1000 feet	1200 feet	1500 feet	1800 feet	
20	167	28	35	42	49	56	64	71	78	92	107	32	37	43	48	54	62	71	38	42	46	53	60	**20**
25	187	35	44	53	62	71	79	88	97	115	133	40	46	53	60	67	77	87	45	50	55	63	70	**25**
30	205	42	52	63	73	84	95	105	116	137	158	47	55	63	71	79	91	103	53	59	65	74	82	**30**
35	221	49	61	73	85	97	110	122	134	158	183	55	65	74	83	93	107	121	62	69	76	86	96	**35**
40	237	55	69	83	96	110	124	138	151	179	206	63	73	84	95	105	121	137	70	78	86	97	108	**40**
45	251	62	77	93	108	123	139	154	169	200	230	70	82	94	106	118	135	153	79	87	95	108	121	**45**
50	265	69	86	103	120	137	154	171	188	222	256	78	91	104	117	130	150	169	88	98	107	121	135	**50**
55	277	76	94	112	131	149	168	186	204	241	278	86	100	114	128	142	164	185	96	107	117	132	147	**55**
60	290	83	103	123	143	163	183	203	223	263	304	93	109	124	139	155	178	201	105	116	127	143	160	**60**
65	301	89	111	132	154	175	197	218	240	283	326	101	117	134	151	167	192	217	114	126	138	156	174	**65**
70	313	96	119	142	165	188	211	234	257	303	....	108	126	144	162	180	206	233	122	135	148	167	186	**70**
75	324	103	128	152	177	202	227	252	276	325	....	116	135	154	173	192	221	249	130	144	157	178	198	**75**
80	335	110	136	162	188	215	241	267	294	....	....	124	144	165	185	206	236	267	138	153	167	189	210	**80**
85	345	116	144	171	199	226	254	282	309	....	....	131	153	174	195	217	249	281	147	163	178	201	224	**85**
90	355	123	152	181	210	240	269	298	327	....	....	139	161	184	207	229	263	297	156	172	188	212	237	**90**
95	365	130	160	191	222	252	283	314	....	....	....	146	170	194	218	242	277	313	164	181	198	224	249	**95**
100	374	136	168	201	233	265	297	329	....	....	....	154	178	203	228	253	290	....	172	190	208	235	261	**100**

1¼-INCH SMOOTH NOZZLE. — 2½- AND 3-INCH HOSE.

Nozzle pressure indicated by Pitot gauge	Discharge, gallons, per minute	Pressures required at hydrant or fire engine, while stream is flowing, to maintain nozzle pressures given in first column, through various lengths of best quality 2½- and 3-inch rubber-lined hose																							Nozzle pressure indicated by Pitot gauge
		Single 2½-inch lines										Single 3-inch lines							Two 2½-inch lines Siamesed						
		100 feet	200 feet	300 feet	400 feet	500 feet	600 feet	700 feet	800 feet	1000 feet	1200 feet	400 feet	600 feet	800 feet	1000 feet	1200 feet	1500 feet	1800 feet	600 feet	800 feet	1000 feet	1200 feet	1500 feet	1800 feet	
20	206	32	42	53	64	75	85	96	107	128	149	37	46	54	62	70	83	95	39	45	51	57	67	76	**20**
25	230	40	53	66	79	92	105	118	131	158	184	47	57	67	77	87	102	117	48	55	62	70	80	91	**25**
30	253	48	63	79	95	110	126	142	157	189	220	56	68	81	93	105	123	142	57	66	74	83	96	109	**30**
35	273	55	73	91	109	127	145	163	181	217	256	65	79	92	106	120	141	161	66	76	86	95	110	125	**35**
40	292	63	83	104	124	144	165	185	206	246	287	74	89	105	120	136	159	183	75	87	99	110	127	144	**40**
45	309	70	93	116	138	161	183	206	229	274	319	83	100	117	135	152	178	204	84	96	109	121	140	158	**45**
50	326	78	103	128	153	178	203	228	253	303	....	91	111	130	149	168	197	226	93	107	121	135	155	176	**50**
55	342	86	113	140	167	194	222	249	276	330	....	100	121	142	163	184	216	247	102	117	132	147	169	192	**55**
60	357	93	123	152	182	211	241	270	300	....	....	109	132	155	178	201	235	270	111	128	144	160	185	210	**60**
65	372	101	133	164	196	228	260	292	323	....	....	118	143	167	192	217	254	291	120	137	155	173	199	225	**65**
70	386	108	142	176	210	244	278	312	....	....	....	127	154	180	206	233	272	....	129	147	166	185	213	241	**70**
75	399	116	152	188	224	261	297	333	....	....	....	136	164	192	220	248	290	....	137	157	177	197	227	257	**75**
80	413	124	163	201	240	279	318	....	....	....	....	145	175	205	235	265	....	....	147	169	190	212	244	276	**80**
85	425	131	172	213	254	295	....	....	....	....	....	152	184	216	247	279	....	....	156	179	201	224	258	292	**85**
90	438	139	182	225	269	312	....	....	....	....	....	162	195	228	261	295	....	....	165	189	213	237	273	309	**90**
95	449	146	191	236	282	327	....	....	....	....	....	170	205	240	275	....	....	....	173	198	223	248	286	323	**95**
100	461	153	201	248	295	....	....	....	....	....	....	179	215	252	288	....	....	....	182	208	235	261	300	....	**100**

1⅜-INCH SMOOTH NOZZLE. — 2½- AND 3-INCH HOSE

Nozzle pressure indicated by Pitot gauge	Discharge, gallons per minute	Pressures required at hydrant or fire engine, while stream is flowing, to maintain nozzle pressures given in first column, through various lengths of best quality 2½- and 3-inch rubber-lined hose																						Nozzle pressure indicated by Pitot gauge
		Single 2½-inch lines								Single 3-inch lines							Two 2½-inch lines Siamesed							
		100 feet	200 feet	300 feet	400 feet	500 feet	600 feet	700 feet	800 feet	200 feet	400 feet	600 feet	800 feet	1000 feet	1200 feet	1500 feet	400 feet	600 feet	800 feet	1000 feet	1200 feet	1500 feet	1800 feet	
20	250	37	52	68	83	98	113	128	144	34	45	57	68	80	92	109	37	46	54	63	71	84	96	**20**
25	280	46	64	83	102	121	139	158	177	41	56	70	85	99	113	135	46	57	67	78	88	104	119	**25**
30	307	55	77	99	121	144	166	188	210	50	67	84	101	118	135	161	56	68	81	93	106	124	143	**30**
35	331	64	89	115	140	166	191	217	242	58	78	97	117	137	157	187	65	80	94	108	122	143	165	**35**
40	354	73	102	131	160	189	218	247	276	67	89	112	134	157	180	214	74	90	106	122	138	162	186	**40**
45	376	81	114	146	178	211	243	275	307	74	99	125	150	175	200	238	83	101	119	137	155	182	209	**45**
50	396	90	125	161	196	222	257	293	328	82	109	137	164	192	220	267	92	111	131	151	171	201	230	**50**
55	415	99	137	176	215	254	292	331	....	90	121	151	182	212	242	288	100	122	144	165	187	219	252	**55**
60	434	107	149	191	233	276	318	....	....	98	131	163	196	229	262	....	109	133	156	180	203	238	273	**60**
65	451	116	161	206	251	297	....	....	....	106	141	177	212	247	282	....	118	143	168	194	219	257	294	**65**
70	469	125	173	222	270	319	....	....	....	114	152	189	227	265	303	....	128	155	182	209	236	277	317	**70**
75	485	134	185	237	289	....	....	....	....	122	162	203	243	283	....	....	137	165	194	223	252	295	....	**75**
80	500	142	196	251	305	....	....	....	....	130	172	215	257	300	....	....	145	175	206	236	266	312	....	**80**
85	516	151	209	267	325	....	....	....	....	138	183	229	274	....	....	....	153	186	218	250	282	331	....	**85**
90	531	159	220	281	....	....	....	....	....	146	194	241	289	....	....	....	162	196	230	264	298	....	....	**90**
95	546	168	232	297	....	....	....	....	....	153	203	254	304	....	....	....	170	206	241	277	313	....	....	**95**
100	560	177	244	312	....	....	....	....	....	162	215	267	....	....	....	....	179	217	254	291	329	....	....	**100**

1½-INCH SMOOTH NOZZLE. — 2½- AND 3-INCH HOSE

Nozzle pressure indicated by Pitot gauge	Dis-charge, gallons per min-ute	Pressures required at hydrant or fire engine, while stream is flowing, to maintain nozzle pressures given in first column, through various lengths of best quality 2½- and 3-inch rubber-lined hose																							Nozzle pressure indicated by Pitot gauge
		Single 2½-inch lines								Single 3-inch lines							Two 2½-inch lines Siamesed								
		100 feet	200 feet	300 feet	400 feet	500 feet	600 feet	700 feet	800 feet	200 feet	400 feet	600 feet	800 feet	1000 feet	1200 feet	1500 feet	200 feet	400 feet	600 feet	800 feet	1000 feet	1200 feet	1500 feet	1800 feet	
20	298	44	65	86	107	128	149	170	191	39	55	71	87	104	120	144	33	45	56	68	79	91	108	126	**20**
25	333	54	80	106	132	158	184	210	236	48	68	88	108	128	148	178	41	56	70	84	99	113	135	156	**25**
30	365	65	95	126	157	188	219	250	280	58	81	105	129	153	177	212	49	66	83	100	117	134	160	185	**30**
35	394	75	110	145	181	216	251	287	322	67	94	122	149	177	204	245	57	77	96	116	135	155	184	214	**35**
40	422	85	126	166	206	246	286	327	....	76	107	139	170	201	232	279	65	88	110	132	155	177	211	244	**40**
45	447	96	141	185	230	275	320	....	....	85	120	155	189	224	258	....	73	97	122	146	171	196	233	269	**45**
50	472	106	155	205	254	304	....	....	....	95	133	171	209	247	286	....	81	108	136	163	190	218	259	300	**50**
55	494	116	170	224	278	332	....	....	....	104	145	187	228	270	....	....	88	118	148	178	208	237	282	327	**55**
60	517	126	184	242	301	....	....	....	....	113	158	203	248	293	....	....	96	128	161	193	225	257	305	....	**60**
65	537	136	198	261	324	....	....	....	....	122	170	218	267	....	....	....	104	139	174	208	243	278	....	....	**65**
70	558	146	213	281	....	....	....	....	....	131	183	235	287	....	....	....	112	149	186	223	261	298	....	....	**70**
75	578	156	228	299	....	....	....	....	....	140	196	251	307	....	....	....	120	160	199	239	279	319	....	....	**75**
80	596	166	242	318	....	....	....	....	....	149	208	267	....	....	....	....	127	170	212	254	296	....	....	....	**80**
85	614	176	257	337	....	....	....	....	....	158	220	282	....	....	....	....	135	179	224	268	313	....	....	....	**85**
90	633	187	272	....	....	....	....	....	....	167	233	298	....	....	....	....	143	190	237	284	....	....	....	....	**90**
95	650	197	286	....	....	....	....	....	....	176	245	314	....	....	....	....	152	201	251	301	....	....	....	....	**95**
100	667	207	300	....	....	....	....	....	....	185	257	....	....	....	....	....	160	212	264	316	....	....	....	....	**100**

$1\frac{5}{8}$-INCH SMOOTH NOZZLE. — $2\frac{1}{2}$- AND 3-INCH HOSE

Nozzle pressure indicated by Pitot gauge	Discharge, gallons per minute	Pressures required at hydrant or fire engine, while stream is flowing, to maintain nozzle pressures given in first column, through various lengths of best quality $2\frac{1}{2}$- and 3-inch rubber-lined hose																				Nozzle pressure indicated by Pitot gauge
		Single 2-inch lines						Single 3-inch lines						Two $2\frac{1}{2}$-inch lines Siamesed								
		100 feet	200 feet	300 feet	400 feet	500 feet	600 feet	200 feet	400 feet	600 feet	800 feet	1000 feet	1200 feet	200 feet	400 feet	600 feet	800 feet	1000 feet	1200 feet	1500 feet	1800 feet	
20	350	52	80	108	136	165	193	46	68	90	112	134	156	37	53	68	84	100	115	139	162	**20**
25	392	65	100	135	170	205	240	57	84	111	138	165	192	47	66	85	104	123	143	171	200	**25**
30	429	77	118	160	201	242	284	68	100	132	164	196	228	56	79	102	125	148	171	205	240	**30**
35	463	89	136	184	231	279	326	78	115	152	189	226	263	65	91	117	144	170	197	236	276	**35**
40	496	101	155	208	262	316	....	89	131	173	215	257	299	74	104	134	164	194	224	269	314	**40**
45	525	113	173	233	293	....	....	100	146	193	239	286	....	82	116	149	182	215	248	298	....	**45**
50	554	125	192	258	324	....	....	111	162	214	265	....	....	91	128	165	202	239	275	331	....	**50**
55	581	137	210	282	....	....	....	121	178	234	290	....	....	100	140	181	221	261	301	....	....	**55**
60	607	149	228	306	....	....	....	132	193	254	....	....	....	109	153	196	240	283	327	....	....	**60**
65	631	162	246	330	....	....	....	143	209	275	....	....	....	118	164	211	258	305	....	....	....	**65**
70	655	173	263	....	....	....	....	153	223	294	....	....	....	126	176	226	276	326	....	....	....	**70**
75	678	184	281	....	....	....	....	163	237	312	....	....	....	135	189	242	295	....	....	....	....	**75**
80	700	197	299	....	....	....	....	174	253	....	....	....	....	144	201	258	314	....	....	....	....	**80**
85	722	209	317	....	....	....	....	184	269	....	....	....	....	153	213	273	....	....	....	....	....	**85**
90	743	220	....	....	....	....	....	195	284	....	....	....	....	162	225	289	....	....	....	....	....	**90**
95	763	232	....	....	....	....	....	205	299	....	....	....	....	170	237	303	....	....	....	....	....	**95**
100	783	244	....	....	....	....	....	216	314	....	....	....	....	179	249	319	....	....	....	....	....	**100**

$1\frac{3}{4}$-INCH SMOOTH NOZZLE. — $2\frac{1}{2}$- AND 3-INCH HOSE

Nozzle pressure indicated by Pitot gauge	Discharge, gallons per minute	Pressures required at hydrant or fire engine, while stream is flowing, to maintain nozzle pressures given in first column, through various lengths of best quality $2\frac{1}{2}$- and 3-inch rubber-lined hose																				Nozzle pressure indicated by Pitot gauge
		Single $2\frac{1}{2}$-in. lines				Single 3-inch lines								Two $2\frac{1}{2}$-inch lines Siamesed									
		100 feet	200 feet	300 feet	400 feet	100 feet	200 feet	300 feet	400 feet	500 feet	600 feet	800 feet	1000 feet	100 feet	200 feet	300 feet	400 feet	500 feet	600 feet	800 feet	1000 feet	1200 feet	
20	407	63	100	138	175	40	55	71	86	101	116	147	177	33	43	53	64	74	84	105	125	146	**20**
25	455	77	123	169	215	49	67	84	102	120	138	173	209	40	53	65	78	91	103	128	154	179	**25**
30	498	91	145	199	253	58	79	100	121	142	163	205	247	49	64	79	94	110	125	155	185	215	**30**
35	538	106	169	231	294	68	92	117	141	166	190	239	288	56	74	91	109	126	143	178	213	248	**35**
40	575	120	191	262	333	77	104	132	159	187	215	270	325	64	84	103	123	143	162	201	241	280	**40**
45	609	135	215	294	....	87	118	149	180	211	241	303	....	73	95	117	139	161	183	227	271	315	**45**
50	643	150	237	325	....	96	130	164	199	233	267	....	....	80	104	128	152	177	201	249	297	....	**50**
55	674	164	259	....	....	105	142	179	216	254	291	....	....	88	114	140	167	193	219	272	324	....	**55**
60	704	177	280	....	....	114	154	194	234	274	314	....	....	96	125	153	182	210	239	296	....	....	**60**
65	732	191	302	....	....	123	166	209	252	296	....	....	....	104	134	165	195	226	257	318	....	....	**65**
70	761	206	325	....	....	133	180	227	273	....	....	....	....	111	144	177	210	243	275	....	....	....	**70**
75	787	220	....	....	....	143	192	242	291	....	....	....	....	118	153	188	223	258	293	....	....	....	**75**
80	813	234	....	....	....	152	204	257	309	....	....	....	....	127	164	201	239	276	313	....	....	....	**80**
85	838	247	....	....	....	160	215	270	....	....	....	....	....	135	174	214	253	293	....	....	....	....	**85**
90	862	261	....	....	....	169	228	286	....	....	....	....	....	142	183	225	266	308	....	....	....	....	**90**
95	885	274	....	....	....	178	240	301	....	....	....	....	....	150	194	237	281	....	....	....	....	....	**95**
100	909	....	....	....	....	188	253	317	....	....	....	....	....	158	204	250	296	....	....	....	....	....	**100**

2-INCH SMOOTH NOZZLE. — 2½- AND 3-INCH HOSE

Nozzle pressure indicated by Pitot gauge	Discharge, gallons per minute	Pressures required at hydrant or fire engine, while stream is flowing, to maintain nozzle pressures given in first column, through various lengths of best quality 2½- and 3-inch rubber-lined hose																		Nozzle pressure indicated by Pitot gauge
		Single 2½-inch lines			Single 3-inch lines							Two 2½-inch lines Siamesed								
		100 feet	200 feet	300 feet	100 feet	200 feet	300 feet	400 feet	500 feet	600 feet	800 feet	100 feet	200 feet	300 feet	400 feet	500 feet	600 feet	800 feet	1000 feet	
20	532	90	152	214	52	76	100	124	148	172	220	41	58	75	92	110	127	161	195	**20**
25	594	111	187	263	65	94	123	152	182	211	270	51	72	93	114	135	156	198	240	**25**
30	651	132	222	312	77	112	147	181	216	251	321	61	86	110	135	160	185	234	284	**30**
35	703	152	255	.....	89	129	169	209	249	289	.....	71	100	128	157	186	214	271	329	**35**
40	752	173	290	.....	102	147	193	238	283	.....	.....	81	113	146	178	211	243	308	.....	**40**
45	797	193	323	.....	113	163	213	263	314	.....	.....	90	126	162	198	234	270	.....	.....	**45**
50	841	214	.....	.....	126	182	237	293	.....	.....	.....	100	140	180	220	260	300	.....	.....	**50**
55	881	.....	.....	.....	138	199	260	321	.....	.....	.....	110	153	197	240	284	.....	.....	.....	**55**
60	920	.....	.....	.....	150	216	282	.....	.....	.....	.....	119	166	213	260	308	.....	.....	.....	**60**
65	958	.....	.....	.....	162	233	304	.....	.....	.....	.....	129	180	230	281	.....	.....	.....	.....	**65**
70	994	.....	.....	.....	175	251	327	.....	.....	.....	.....	139	193	248	302	.....	.....	.....	.....	**70**
75	1029	.....	.....	.....	187	268	.....	.....	.....	.....	.....	148	206	264	322	.....	.....	.....	.....	**75**
80	1063	.....	.....	.....	199	285	.....	.....	.....	.....	.....	158	219	280	.....	.....	.....	.....	.....	**80**
85	1095	.....	.....	.....	211	302	.....	.....	.....	.....	.....	167	232	297	.....	.....	.....	.....	.....	**85**
90	1128	.....	.....	.....	223	319	.....	.....	.....	.....	.....	177	245	314	.....	.....	.....	.....	.....	**90**
95	1158	.....	.....	.....	235	335	.....	.....	.....	.....	.....	186	258	.....	.....	.....	.....	.....	.....	**95**
100	1189	.....	.....	.....	247	.....	.....	.....	.....	.....	.....	196	272	.....	.....	.....	.....	.....	.....	**100**

INDEX

EDDY

VALVES AND HYDRANTS

The Eddy Fire Hydrants have many points which should appeal to the progressive water works superintendent. For instance, in it a damaged valve can be replaced in a few minutes and at a slight expense. And note the other features:

Should the standpipe be broken, not a drop of water need be lost.
Positively all water hammer is eliminated.
Hydrant stems cannot be bent.
Nothing can lodge on the valve seat to prevent closing of the valve.
The Drip Rod can be removed without interfering with the hydrant in any manner.
Absolutely frost proof.
The positoin of the stem of the hydrant indicates accurately the position of the valve.
Whatever can pass the valve opening will pass the standpipe.
No frost case is necessary.
All of these points are explained thoroughly in our catalogue, which will pay you to study.

THE EDDY VALVE

The Eddy Valve, too, has its distinctive and individual features. For instance, the double disc gates are free to adjust themselves in different positions every time the valve is closed, thereby keeping smooth and tight faces.

And two bronze hooks on the ball loosely engage with the gates to prevent them from spreading at the top.

The center bearing gates are forced to their seats with equal pressure at all points.

There is a good deal more worth knowing about Eddy Valves. Further details will be sent for the asking.

OTHER EDDY PRODUCTS

The Eddy line of water works products also includes Horizontal and Vertical Check Valves, Foot Valves, Hydraulically Operated Valves, Indicator Posts, Sprinkler Cart Cranes with Automatic Drip Valves, Etc.

All goods made by the Eddy Valve Company are manufactured exclusively at Waterford, New York

EDDY VALVE COMPANY

WATERFORD, N. Y., U. S. A.